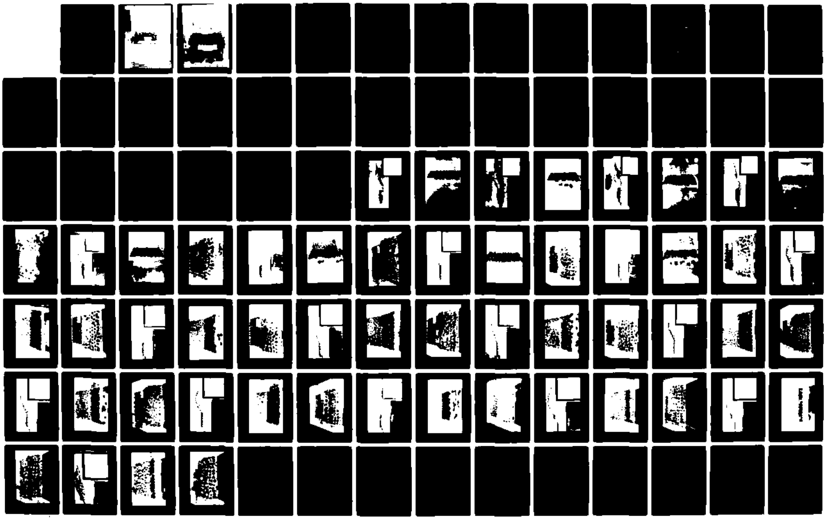
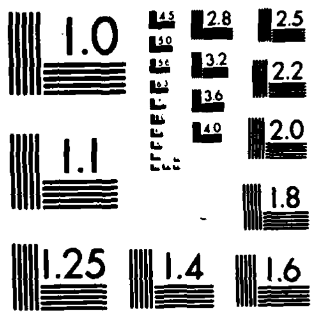
AD-A123 754 REVETMENT STABILITY STUDY FORT FISHER STATE HISTORIC SITE NORTH CAROLINA..(U) ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MS HYDRA.. D G MARKLE 1/2

UNCLASSIFIED NOV 82 WES/TR/HL-82-26 F/G 13/2 NL




1.0
2.8
2.5
3.2
2.2
3.6
4.0
2.0
1.1
1.8
1.25
1.4
1.6

Unclassified

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE

READ INSTRUCTIONS BEFORE COMPLETING FORM

1. REPORT NUMBER Technical Report HL-82-26	2. GOVT ACCESSION NO. AD-A123 754	3. RECIPIENT'S CATALOG NUMBER
4. TITLE (and Subtitle) REVETMENT STABILITY STUDY, FORT FISHER STATE HISTORIC SITE, NORTH CAROLINA; Hydraulic Model Investigation		5. TYPE OF REPORT & PERIOD COVERED Final report 6. PERFORMING ORG. REPORT NUMBER
7. AUTHOR(s) Dennis G. Markle		8. CONTRACT OR GRANT NUMBER(s)
9. PERFORMING ORGANIZATION NAME AND ADDRESS U. S. Army Engineer Waterways Experiment Station Hydraulics Laboratory P. O. Box 631, Vicksburg, Miss. 39180		10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS
11. CONTROLLING OFFICE NAME AND ADDRESS U. S. Army Engineer District, Wilmington P. O. Box 1890 Wilmington, N. C. 28402		12. REPORT DATE November 1982 13. NUMBER OF PAGES 93
14. MONITORING AGENCY NAME & ADDRESS(if different from Controlling Office)		15. SECURITY CLASS. (of this report) Unclassified 15a. DECLASSIFICATION/DOWNGRADING SCHEDULE

16. DISTRIBUTION STATEMENT (of this Report)

Approved for public release; distribution unlimited.

17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)

18. SUPPLEMENTARY NOTES

Available from National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22151

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

Armor stone	Overtopping
Fort Fisher	Revetments
Gabions	Shore protection
Hydraulic models	

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

A hydraulic model investigation was conducted using a two-dimensional (2-D) stability model at an undistorted linear scale of 1:24 (model to prototype). The purposes of the stability tests were as follows:

a. Develop both special- and random-placed armor-stone revetment designs that will be stable for storm conditions which would generate depth-limited breaking waves at the -4.0 ft National Geodetic Vertical Datum (NGVD) contour for a still-water level (swl) of +10.7 ft NGVD.

(Continued)

DD FORM 1 JAN 73 1473 EDITION OF 1 NOV 65 IS OBSOLETE

Unclassified

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

20. ABSTRACT (Continued):

b. Determine the stability response of all plans to depth-limited breaking waves at the -4.0 ft contour for swl's of +8.0 and +5.0.

c. Check the stabilities of the optimum designs (as determined from the model tests and selected by the Wilmington District) when exposed to depth-limited breaking waves at the -4.0 ft contour for swl's of +13.0 and +15.0.

Test results indicated that 5,900-lb, special-placed and 8,600-lb, random-placed armor stones would be stable for the depth-limited breaking wave conditions produced at the +10.7 swl. The 5,900-lb, special-placed armor-stone design was tested on revetments that had a 30-ft-wide gabion toe and a 14-ft-wide Sta-Pod toe and proved to be stable for both cases. The 8,600-lb, random-placed armor-stone design was only tested on the revetment with the 30-ft-wide gabion toe. The 30-ft-wide toe incorporated a six-row gabion blanket and the 14-ft-wide toe used 8,919-lb Sta-Pods as buttressing on the outer sea-side toe. All of the acceptable designs showed slightly higher degrees of damage, but none of them failed when exposed to depth-limited breaking waves at the +13.0 and +15.0 swl's. None of the designs exhibited any additional stability problems when exposed to the depth-limited, breaking waves at the +8.0 and +5.0 swl's.

PREFACE

In October 1976, Congress authorized the design and construction of a beach erosion control project to protect the Fort Fisher State Historic Site and the immediate vicinity surrounding the site. As part of this project, a series of two-dimensional stability model tests were conducted to determine a stable design for the proposed armor-stone shoreline revetment.

The model investigation reported herein was initially requested by the U. S. Army Engineer District, Wilmington (SAW), in a letter to the U. S. Army Engineer Waterways Experiment Station (WES) dated 23 June 1981. Funding authorization by SAW was granted in SAW Intra-Army Order No. PC-81-319, dated 5 August 1981.

Model tests of various revetment designs were conducted at WES during the period October 1981 to March 1982 under the general direction of Mr. H. B. Simmons, Chief of the Hydraulics Laboratory, Dr. R. W. Whalin, Chief of the Wave Dynamics Division, and Mr. D. D. Davidson, Chief of the Wave Research Branch. Tests were conducted by Mr. M. S. Taylor, Engineering Technician, assisted by Messrs. C. R. Herrington and C. Lewis, Engineering Technicians, and Mrs. B. J. Wright, Engineering Aide, under the supervision of Mr. D. G. Markle, Research Hydraulic Engineer. This report was prepared by Mr. Markle.

Liaison was maintained during the course of the investigation by means of conferences, progress reports, and telephone conversations.

Commander and Director of WES during the conduct of this study and the preparation and publication of this report was COL Tilford C. Creel, CE. Technical Director was Mr. F. R. Brown.

A

CONTENTS

CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
feet	0.3048	metres
pounds (force)	4.448222	newtons
pounds (force) per cubic foot	157.087467	newtons per cubic metre
miles (U. S. statute)	1.609344	kilometres

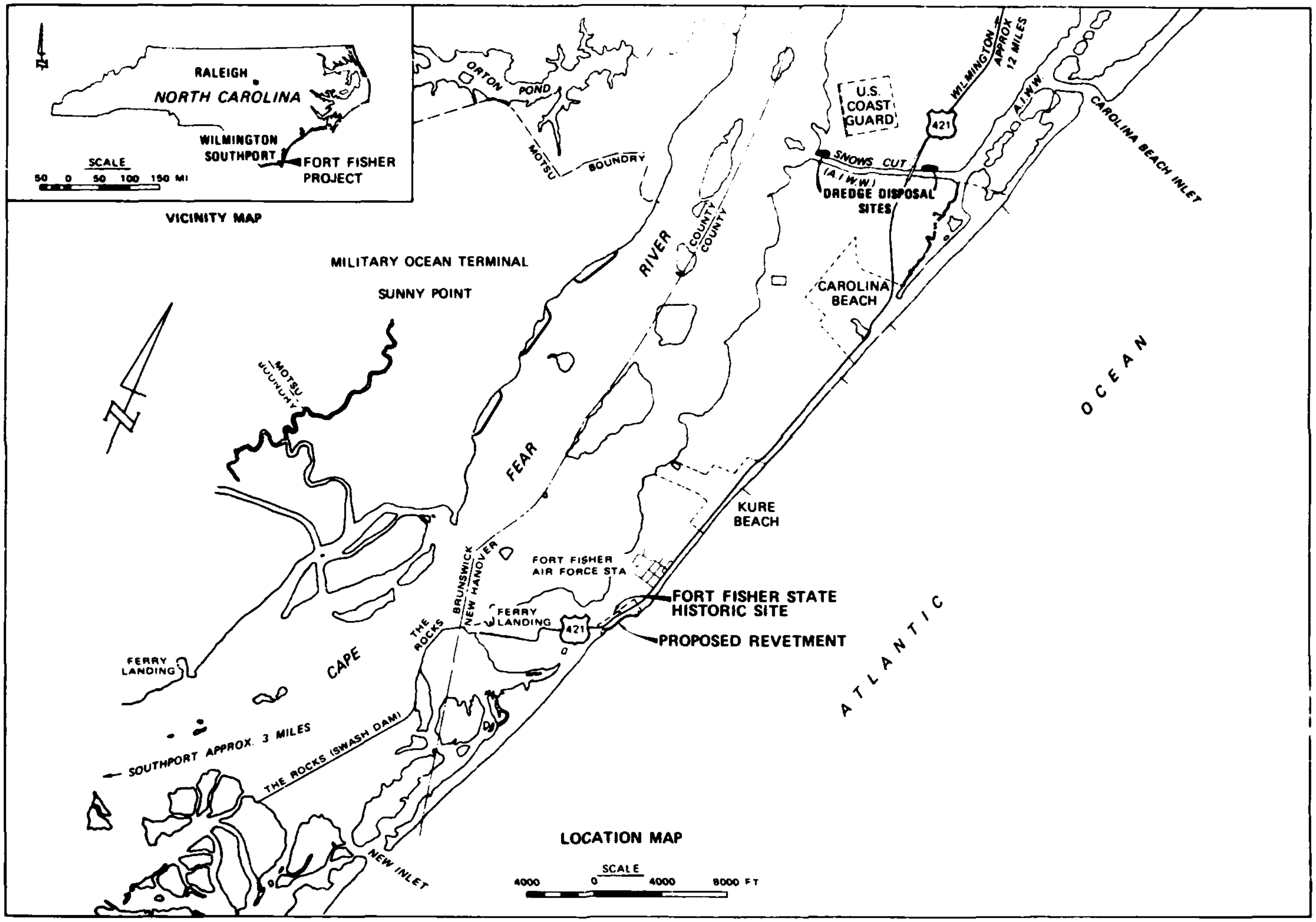


Figure 1. Project vicinity and location maps

REVETMENT STABILITY STUDY

FORT FISHER STATE HISTORIC SITE, NORTH CAROLINA

Hydraulic Model Investigation

PART I: INTRODUCTION

The Prototype

1. Fort Fisher State Historic Site is located in New Hanover County, about 20 miles* south of Wilmington, North Carolina, on the peninsula that separates the lower Cape Fear River from the Atlantic Ocean (Figure 1). Fort Fisher, constructed in 1862, was the largest Civil War earthwork fortification in the Confederacy. These fortifications were important for the South because they kept the Port of Wilmington open until the last few months of the Civil War.

The Problem

2. Severe beach erosion and shoreline retreat have produced extensive damage at the site. A rapid retreat of the shoreline began in the 1940's. Rock outcroppings along the northeast shoreline of the site appear to have interrupted the north to south transport of beach sand and resulted in loss of the protective beach. This has allowed storm waves to impinge directly on the shoreline bluffs, resulting in rapid recession of the shoreline.

Proposed Protective Structure

3. A rubble-mound revetment, 3,200 ft long, has been proposed to protect the eroding shoreline at the historic site (Figure 2). The

---

* A table of factors for converting U. S. customary units of measurement to metric (SI) is presented on page 3.

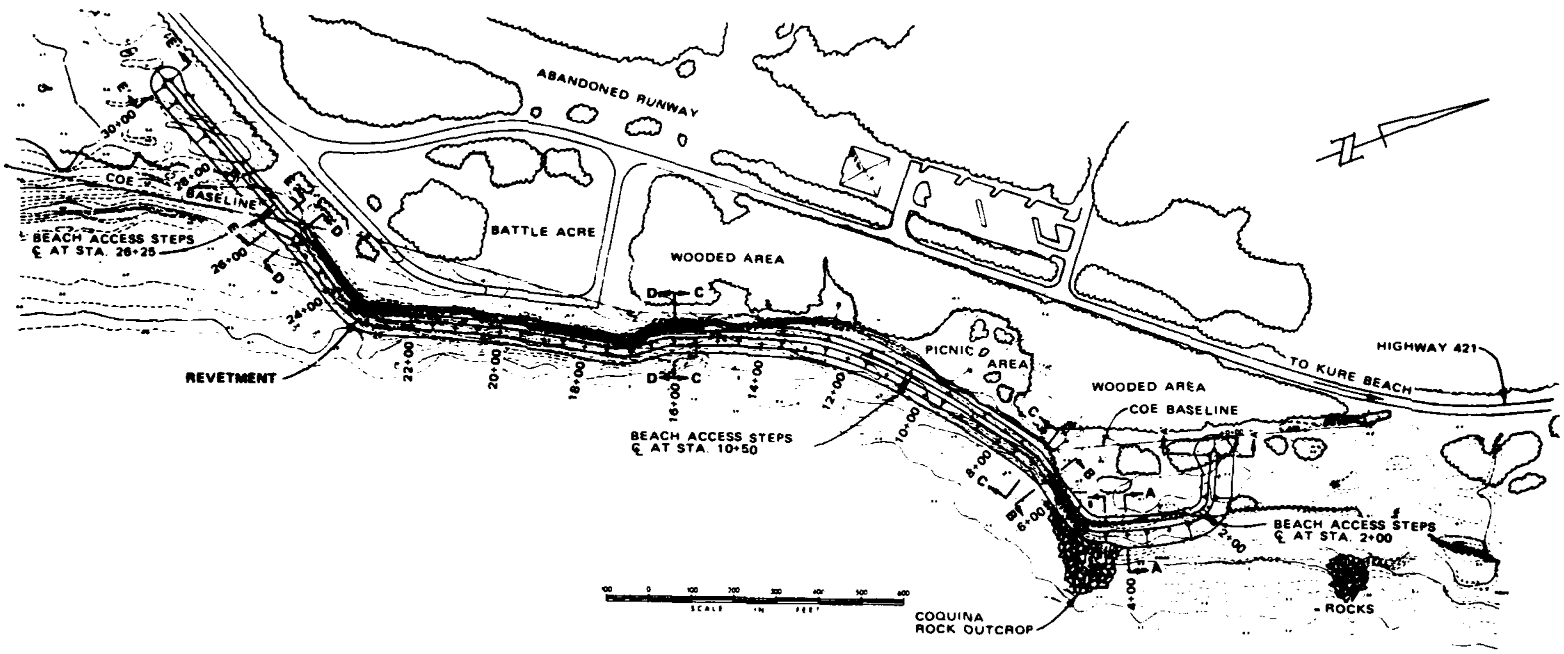


Figure 2. Proposed location and alignment of armor-stone revetment

revetment will extend up to a crown elevation of +13.0 ft NGVD,*,** and tiebacks are planned for both the southern and northern limits of the revetment.

Purposes of the Model Study

4. At the request of the U. S. Army Engineer District, Wilmington (SAW), two-dimensional (2-D) revetment stability tests were conducted by the U. S. Army Engineer Waterways Experiment Station (WES). The purposes of these wave stability tests were as follows:

- a. Develop both special- and random-placed armor-stone designs that will be stable for storm conditions which would generate depth-limited breaking waves at the -4.0 ft contour for a still-water level (swl) of +10.7.
- b. Determine the stability response of all plans to depth-limited breaking waves at the -4.0 ft contour for swl's of +8.0 and +5.0.
- c. Check the stability of the optimum designs (as determined from the model tests and selected by SAW) when exposed to depth-limited breaking waves at the -4.0 ft contour for swl's of +13.0 and +15.0.

Preliminary designs of two alternative revetment plans were prepared by SAW (Figure 3) and furnished to WES. These designs were used as a starting point for the test series reported herein.

---

* For convenience, symbols and unusual abbreviations are listed and defined in the Notation (Appendix A).

** All elevations (el) cited herein are in feet referred to the National Geodetic Vertical Datum (NGVD).

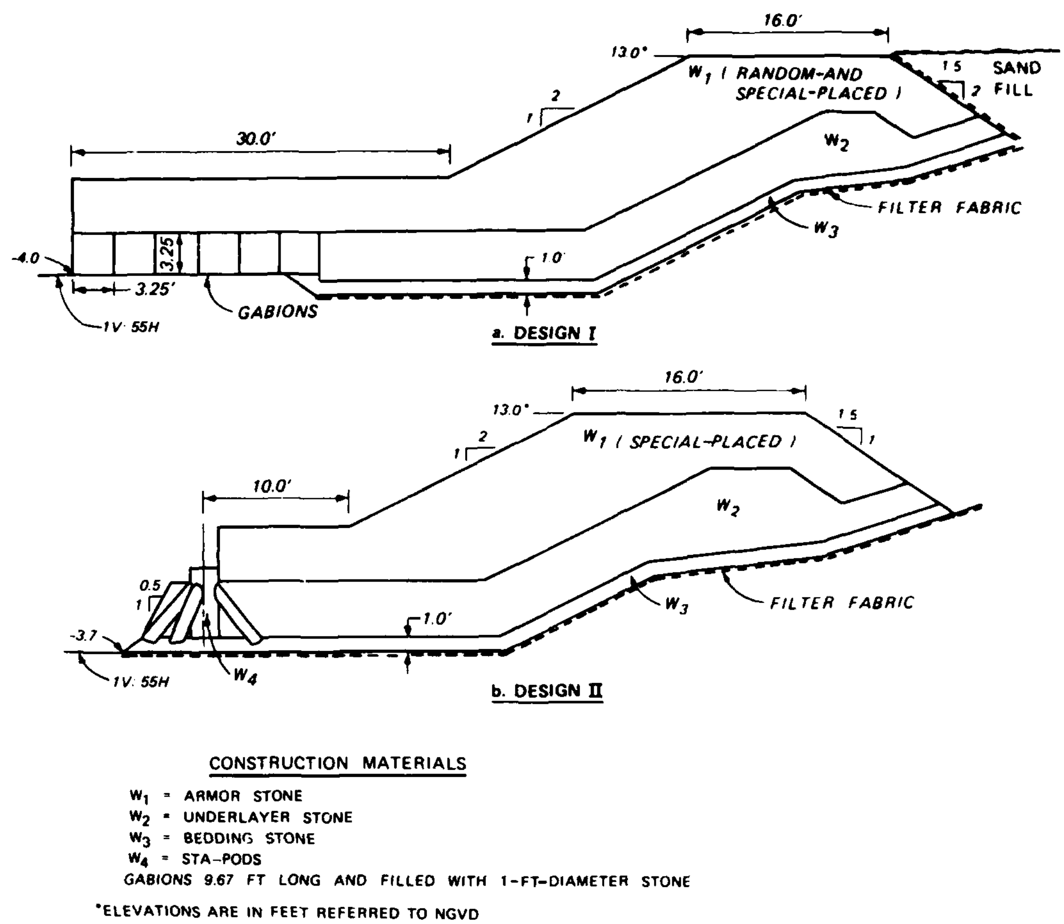


Figure 3. Alternative revetment designs

PART II: THE MODEL

Test Facilities and Equipment

5. All tests were conducted in a 2-ft-wide and 165-ft-long flume in which the depth varied from 4.5 ft in the test area to 6.5 ft at the wave paddle (Figure 4). The flume was equipped with a flap-type wave generator capable of producing monochromatic waves of various periods and heights.

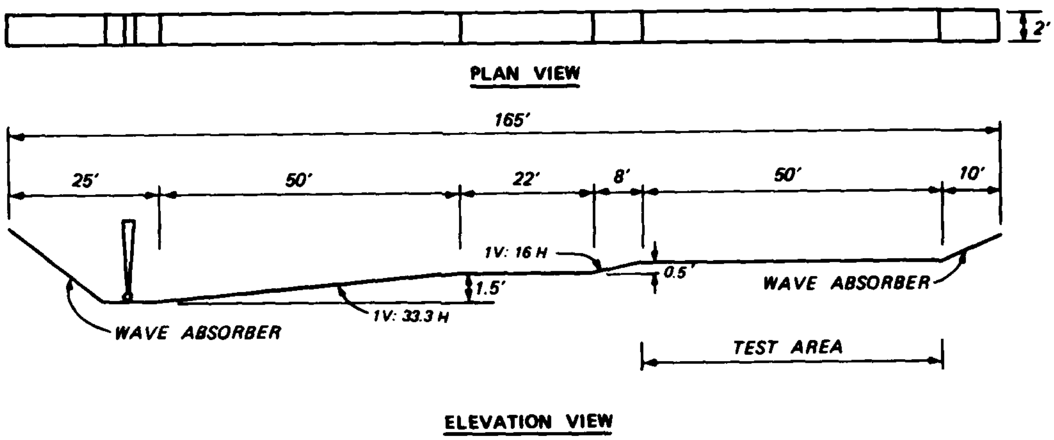


Figure 4. Test flume geometry

Design of Model

6. The 2-D wave stability tests, with incident wave crests parallel to the longitudinal axis of the revetment, were conducted at an undistorted linear scale of 1:24, model to prototype. Scale selection was based on the size of model armor stone relative to the size of armor stone proposed for use on the prototype revetment, elimination of stability scale effects,* and capabilities of the available wave flume.

---

* R. Y. Hudson. 1975 (Jun). "Reliability of Rubble-Mound Breakwater Stability Models," Miscellaneous Paper H-75-5, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

Based on Froude's model law* and the linear scale of 1:24, the following model to prototype relations were derived. Dimensions are in terms of length (L) and time (T).

Characteristics	Dimensions	Model to Prototype Scale Relations
Length	L	$L_r = 1:24$
Area	L^2	$A_r = L_r^2 = 1:576$
Volume	L^3	$V_r = L_r^3 = 1:13,824$
Time	T	$T_r = L_r^{1/2} = 1:4.9$

7. The specific weights of water used in the model and of sea-water were assumed to be 62.4 pcf and 64.0 pcf, respectively. The specific weight of the cover-layer armor stone in the model was identical with its prototype counterpart. Based on this information, the following transference equation was used to calculate the cover-layer armor-stone weight for the 1:24-scale model:

$$\frac{(W_a)_m}{(W_a)_p} = \frac{(\gamma_a)_m}{(\gamma_a)_p}\left(\frac{L_m}{L_p}\right)^3 \left[\frac{(S_a)_p - 1}{(S_a)_m - 1}\right]^3$$

where

subscripts m and p = model and prototype quantities, respectively

W_a = weight of an individual armor stone, lb

γ_a = specific weight of an individual armor stone, pcf

L_m/L_p = linear scale of the model

S_a = specific gravity of an individual armor stone relative to the water in which the revetment is constructed, i.e., $S_a = \gamma_a/\gamma_w$

γ_w = specific weight of water, pcf

* J. C. Stevens et al. 1942. "Hydraulic Models," Manual of Engineering Practice No. 25, American Society of Civil Engineers, New York.

8. The specific weight of the prototype underlayer, gabion fill, and bedding stone was 125 pcf and that of the model material was 165 pcf. Due to the large difference in the specific weights, use of the transference equation described in paragraph 7 would have resulted in model-to-prototype stone layer thickness ratios that were much smaller than called for by the 1:24 linear scale. This would have required the use of three to four model stone layers to meet the proper scaled layer thickness of two layers of prototype stone. The model tests were not addressing the stability against wave attack of the underlayer, gabion, and bedding stones; therefore, to maintain the correct model-to-prototype stone layer thickness ratios, these materials were scaled down geometrically using the following equation:

$$(\ell_a)_m = k_\Delta \left[\frac{(W_a)_p}{(\gamma_a)_p} \right]^{1/3} \frac{L_m}{L_p}$$

where

ℓ_a = characteristic length of armor stone, ft

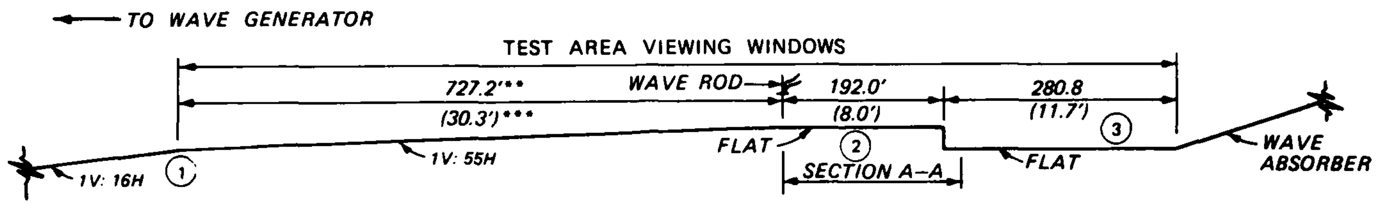
k_Δ = layer coefficient (k_Δ = 1.15 for rough quarrystone)

This geometric scaling reproduced the correct layer thickness for the beddings and underlayers on all plans but resulted in individual stone weights that were too large. The individual weights of the model bedding and underlayer materials listed in the plates for each plan are the actual weights that were used in the model. The prototype weights are the recommended weights and are not the actual weights that were represented in the model.

Model Construction and Test Procedures

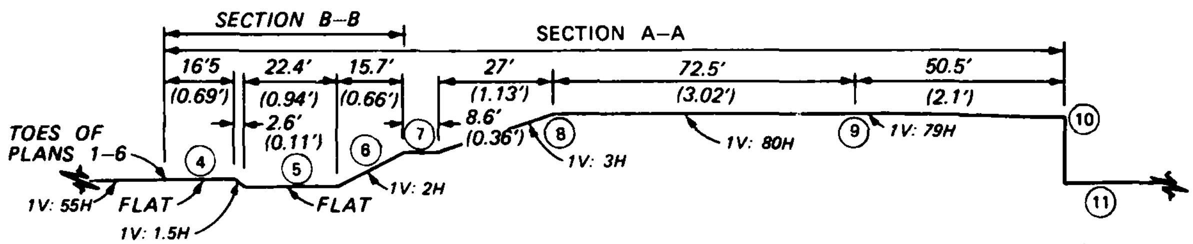
Modeling local topography and bathymetries and flume calibration

9. The topography and bathymetries shown in Figure 5 were



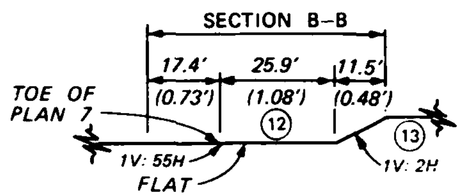
a. TEST AREA GEOMETRY AND WAVE ROD LOCATION DURING CALIBRATION

ELEVATION



b. GEOMETRY OF SECTION A-A DURING CONDUCT OF STABILITY TESTS FOR PLANS 1-6

ELEVATION



c. GEOMETRY OF SECTION B-B DURING CONDUCT OF STABILITY TESTS FOR PLAN 7

ELEVATION

PROTOTYPE ELEVATION FT NGVD

1	-17.2	8	12.1
2	- 4.0	9	13.0
3	-17.2	10	12.4
4	- 4.0	11	-17.2
5	- 5.8	12	- 3.7
6	2.2	13	2.2
7	3.1		

** PROTOTYPE DIMENSIONS
*** MODEL DIMENSIONS

Figure 5. Test area geometry during calibration (a) and conduct of stability tests (b and c)

furnished by SAW and were constructed in the flume test area (Figure 4) for use during flume calibration and conduct of the wave stability tests, respectively. The 1V-on-55H slope was selected as representative of the steepest existing sea floor bathymetry seaward of the -4.0 ft contour at the Fort Fisher project. During calibration, a flat bottom extended landward of the -4.0 ft contour (Area 2, Figure 5a). This enabled the measurement of incident wave heights that were not contaminated with waves reflected off the existing shoreline slopes. Test waves of the required characteristics for the selected test depths were generated by varying the frequency and amplitude of the wave generator paddle. Changes in water-surface elevations were measured by an electrical wave-height gage positioned at the -4.0 ft contour (Figure 5a) and recorded on chart paper by an electrically operated oscillograph. At the completion of the flume calibration, the flat bottom landward of the wave gage (Area 2, Figure 5a) was removed and the shoreline bathymetry and topography, including the needed excavation for construction of the revetment, were installed (Section A-A, Figure 5b). Section B-B of Section A-A (Figures 5b and 5c) was modified before tests were initiated for Plan 7.

Methods of constructing test sections

10. Model revetment sections were constructed to reproduce, as closely as possible, results of prototype construction methods. The bedding layer was dumped by bucket or shovel into the flume and smoothed to grade with hand trowels. The model gabions, constructed of window screen filled with small stones, were placed along the revetment toes of Plans 1-6 (Plates 1-6, respectively), and the underlayer stone was placed and smoothed to grade in the same manner as the bedding stone. A single row of interlocked Sta-Pods was placed over the bedding layer on the sea-side toe of Plan 7 (Plate 7). The underlayer stone was dumped and smoothed to grade landward of the Sta-Pods and a small quantity of underlayer was hand-placed over the bedding stone on the sea side of the Sta-Pods. One layer of armor stone was added to the toe, crown, and landside slope and two layers

were added to the seaside slope of the revetment using either random or special placement, depending on the test plan being constructed. Random placement means that the stones are individually placed, but they are laid down in such a manner that no intentional interlocking or special orientation of the armor stone is achieved. Special placement means individual placement where the stones are specifically oriented and/or fitted to maximize contact between stones. Special placement over the 30-ft-wide gabion toe and the 14-ft-wide Sta-Pod toe consisted of placing the armor stones such that their long axes were horizontal and maximum contact was attained between adjacent stones. Special placement on the crown and slopes was accomplished by placing the long axes of stones perpendicular to the revetment slopes, and the stones were oriented such that maximum contact was achieved between adjacent stones. Where two layers of stones were placed on the sea-side slope, the bottom layer was placed with their long axes either parallel or perpendicular to the slope. All of the stones in the top layer were placed with their long axes perpendicular to the slope. Except during construction of the revetment crown, where special stone shapes were needed to fit into the transition area of the two-layer to one-layer armor-stone crown, no intentional selections of stone shapes were made (i.e., stones were picked out of the stockpile at random).

Selection of test conditions

11. Based on anticipated prototype conditions and available prototype data, SAW decided that the stability tests should consider wave periods of 8, 10, and 12 sec and the worst breaking waves that could be generated in the test flume at the -4.0 ft contour for a design swl of +10.7 and a foreslope of 1V on 55H. After the first revetment test section had been installed in the test flume, it was exposed to a range of wave heights for each of the wave periods at the +10.7 swl. Model observations indicated that the 8- and 10-sec wave periods created the worst breaking wave attack on the toe and sea-side slope. The 12-sec wave period created the worst wave attack on the crown and landside slope armor stone. Based on these observations,

all three wave periods were selected for inclusion in the test conditions of Hydrograph A (Plate 8 and Table 1).

12. All waves included in Hydrograph A are breaking waves, except for the shakedown waves that were used to simulate the more frequently occurring smaller waves which allow some seating and consolidation of the armor stone prior to exposure to the larger test waves. The 8- and 10-sec test waves were best classified as plunging breakers while the 12-sec waves were spilling breakers.

13. All but one of the test plans were exposed to the worst breaking waves that could be generated for wave periods of 8, 10, and 12 sec at swl's of +5.0 and +8.0. These tests were conducted to determine the stability responses of the sea-side toes when exposed to breaking wave conditions at the lower swl's. These test conditions were referred to as Hydrograph B (Plate 9 and Table 1). Hydrograph B tests were conducted following Hydrograph A tests and without reconstructing the test section. Since one plan was unacceptable for Hydrograph A, it was not subjected to Hydrograph B.

14. Two plans (one with a gabion toe and one with a Sta-Pod toe) were exposed to the worst breaking waves that could be generated for wave periods of 8, 10, and 12 sec at swl's of +13.0 and +15.0. Results of these tests gave some indication of the degree of damage that could occur if the design swl and wave conditions were exceeded. These test conditions were referred to as Hydrograph C (Plate 10 and Table 1). Hydrograph C tests also were conducted following Hydrograph A tests without reconstructing the test section.

Model operation

15. Each of the revetment plans was constructed in the test flume, before-test photographs were taken, the test flume was flooded to the appropriate depth, and the plan was exposed to the shakedown and test waves. Prototype test time was accumulated in 30-sec (model time) cycles (i.e., the wave generator was started, run for 30 sec, and then stopped). After each 30-sec cycle, sufficient time was provided for the test flume to still out before the next cycle was run. This procedure eliminated contamination of generated waves by

rereflected waves from the wave generator. During stilling time between cycles, detailed model observations of the structure's response to the previous cycle of test waves were recorded by the model operator. These observations included any movement occurring on the structure and a general statement of the condition of the structure at that point in the test. At the conclusion of the test, the flume was drained and the after-test conditions of the structure were summarized in the test notes and documented with photographs. Each test plan was rebuilt and the test was repeated. The purpose of the repeat test was to determine the presence of uncontrolled variations in model construction that might affect the stability of the structure. The initial and repeat test results were very similar for all plans, except Plan 2, where slightly different construction techniques were used on the initial and repeat test sections. For all tests, except Plan 2, only one of the test results is reported herein. Where differences in damage occurred between the two tests, the test showing the higher degree of damage was selected for reporting herein.

Methods of reporting model
<u>observations and test results</u>

16. The following list of adjectives, in order of increasing severity, was used for recording model observations of armor unit activity and reporting test results for damage on each test section: (a) slight, (b) minor, (c) moderate, (d) significant, (e) major, and (f) extensive. Slight and minor were used to describe acceptable activities or results, moderate described borderline acceptability, while significant to extensive described unacceptable conditions of increasing severity. Use of these adjectives allowed some quantification of the severity and/or amount of rocking in place, onslope displacement, offslope displacement, and resulting damage accrued by the revetment's cover-layer stone. By using the descriptive adjectives and the before- and after-test photographs, comparisons of alternative test plans can be made.

PART III: TESTS AND RESULTS

Development of Plans

17. Seven plans were tested for the armor-stone revetment. Plans 1-6 had 30-ft-wide sea-side toes. The outer 19.5-ft section of the toe armor stone was placed on six rows of single-layered gabions. Plan 7 had a 14-ft-wide sea-side toe; a single row of interlocked Sta-Pods extended along the outer sea-side toe. All plans used armor stone for the primary cover-layer protection. Special armor-stone placement was used on Plans 1, 4, 6, and 7. The remainder of the plans were constructed using random armor-stone placement. Both construction techniques are described in paragraph 10.

Description of Test Plans and Test Results

Plan 1

18. Plan 1 (Plate 1, Photos 1 and 2) was constructed using special-placed armor-stone protection from a -4.0 bottom toe elevation to a crown elevation of +13.0. Stone, with an average individual prototype weight of 2 lb, was used to construct the 1.0-ft bedding layer. The bedding was overlaid with stone weighing an average of 675 lb. The seaward 19.5 ft of the 30-ft-wide revetment toe was constructed by placing six rows of 3.25- × 3.25- × 9.67-ft gabions with their long axes parallel to the revetment crown. The gabions were filled with 1-ft-diam marine limestone and overlaid with one layer of special-placed 8,600-lb armor stone. The remainder of the special-placed toe armor stone overlaid the underlayer stone. Special-placed, 8,600-lb armor stone extend up the 1V-on-2H sea-side slope, over the 16-ft-wide crown, and part way down the 1V-on-1.5H landside slope. Filter fabric was placed over the landside slope and a sand fill was placed landward of the revetment crown.

19. Exposure of Plan 1 to Hydrograph A resulted in minor damage to the outer revetment toe, no damage to the remaining armor-stone cover

layer, and extensive damage to the sand fill (Photos 3 and 4). Three stones were displaced off the seaward edge of the toe during Step 1 of the hydrograph. This was the only armor-stone displacement observed throughout the test. Several of the outermost toe stones rocked in place throughout the test. These stones were not displaced, but their movement was more pronounced during Step 3. Two or three armor stones on the upper sea-side slope and crown rocked in place throughout the test, but no displacement occurred. Erosion of the sand fill started during Step 1 and continued throughout the test. After completing Hydrograph A and the after-test documentation, Plan 1 was exposed to Hydrograph B. These wave and swl conditions caused no additional damage and appeared to be less severe on the revetment stability than the wave and swl conditions of Hydrograph A.

20. It was evident during testing of Plan 1 that the sand fill was very unstable when exposed to the overtopping conditions of Hydrograph A and measures must be taken to stabilize the fill area. The model gave a qualitative indication of sand movement, but it could not be used as a quantitative indicator of movement. Any quantity of fill that remains in place will give added stability to the landside slope. Based on discussions between SAW and WES, the decision was made to delete the sand fill and filter fabric from the remainder of the 2-D stability test series. This allowed for measurement of the landside slope stability without the influence of the added stability that may or may not be provided by the sand fill and filter fabric.

Plan 2

21. Plan 2 (Plate 2 and Photos 5 and 6) was identical with Plan 1 except that random-placed armor stone rather than special-placed armor stone was used for Plan 2. Due to the geometry of the one-layer area of the revetment crown and the size of the armor stone, some difficulties were encountered during construction of the first test section of Plan 2. Several areas of the crown had void areas that were quite large, but not large enough to fit an armor stone into without resulting in several areas of the crown that would greatly exceed the +13.0 crown elevation. For the first testing of Plan 2, these void areas

were left open and this resulted in a very loose crown construction. Rocking in place and landward displacement of crown stones started during the shakedown step of Hydrograph A and continued throughout the remainder of the test. By the end of Step 2, three armor stones had been displaced off of the outer sea-side toe. In-place rocking and uplifting of several of the toe stones continued throughout the remainder of the test, but no additional displacement occurred. During Step 3, one armor stone was displaced from the lower sea-side slope out onto the revetment toe. This displacement did not appear to have any effect on the sea-side slope stability. At the conclusion of Hydrograph A, nine armor stones had been displaced in a landward direction off the crown and landside slope. All significant displacement appeared to have stopped in this area, but there was still some very slow progressive damage occurring on the landside of the revetment at the end of the test. Photos 7-9 show the slight sea-side slope damage, minor sea-side toe damage, and significant crown and landside slope damage accrued by Plan 2 during its first exposure to the test conditions of Hydrograph A.

22. Plan 2 was rebuilt (Photos 10-12) and again exposed to Hydrograph A. Care was taken in selecting stone shapes that would fit into the one-layer area of the revetment crown, and a tighter crown construction was achieved. Exposure to Hydrograph A resulted in minor damage to the sea-side toe (four armor stones displaced) and no damage to the remainder of the revetment (Photos 13-15). Several toe stones rocked in place throughout the test, but no displacement occurred after Step 2. Several stones on the sea-side slope and crown showed a moderate amount of in-place rocking during Step 3; but as stated above, no displacement occurred and all damage to the revetment had subsided well before the end of the test.

23. Without rebuilding the test section, Plan 2 was exposed to Hydrograph B. These wave and swl conditions caused no additional damage and appeared to be less severe on the revetment stability than the wave and swl conditions of Hydrograph A.

<u>Plan 3</u>

24. Tests were initiated for Plan 3 (Plate 3 and Photos 16-18)

to see if 5,900-lb, random-placed armor stone would be stable for the test conditions of Hydrographs A and B. The construction techniques used were identical with those used on the second test section of Plan 2. Except for the weights and layer thicknesses of the armor stone and first underlayer (W_1 and W_2, respectively), Plan 3 was identical with Plan 2. After exposure to Hydrograph A, Plan 3 showed moderate damage to the outer sea-side toe, upper sea-side slope, and crown of the revetment. A small amount of landward slippage of three armor stones on the lower landside slope resulted in minor damage. During Step 1 of the hydrograph, seven armor stones were displaced off the sea-side toe and several other toe stones showed a significant amount of rocking in place. Three additional armor stones, one on the center of the sea-side slope, one on the upper sea-side slope, and one on the crown, were displaced during Step 1. No other significant armor-stone displacement occurred, but several stones on the upper sea-side slope and crown showed moderate amounts of reorientation and rocking in place throughout the remainder of Hydrograph A. Photos 19-21 show the condition of the test section at the end of the test.

25. Without rebuilding the test section, Plan 3 was exposed to Hydrograph B. Except for a moderate amount of rocking in place of several toe stones, no other armor-stone movement was observed on the test section during exposure to the lower water test conditions.

Plan 4

26. Tests were initiated for Plan 4 (Plate 4 and Photos 22-24) to determine the wave stability of special-placed, 5,900-lb armor stone when exposed to the test conditions of Hydrographs A and B. The special-placement techniques used were identical with those used on Plan 1. The overall geometry, size, and layer thicknesses of Plan 4 were identical with Plan 3. During exposure to Hydrograph A, the test section sustained minor damage on the outer sea-side toe and slight damage on the landside slope. Five armor stones were displaced off the sea-side toe and one armor stone was displaced off the lower landside toe. There also was a slight amount of reorientation of three or four armor stones on the landside toe. Except for some minor to moderate

rocking in place of several sea-side toe stones, no other movement was observed during the test, and all damage had stabilized well before the end of the test (Photos 25-27).

27. Plan 4 then was exposed to Hydrograph B. The test section was not rebuilt prior to exposure to these test conditions, and the section showed no change during exposure to these wave and swl conditions.

Plan 5

28. Plan 5 (Plate 5 and Photos 28-30) was constructed and exposed to Hydrograph A to determine the stability response against wave attack of 4,200-lb, random-placed armor stone. By the end of Hydrograph A, the test section had accrued moderate sea-side toe damage (six armor stones displaced downslope), minor sea-side slope damage (two armor stones displaced downslope), and significant crown damage (four armor stones displaced to the toe of landside slope and several additional stones displaced from the sea-side toward the landside of the crown). Armor stone on the sea-side toe, sea-side slope, and crown showed minor to significant rocking in place and reorientation throughout the test. The displacement and in-place armor-stone movement on the crown resulted in several spot lowerings in the original +13.0 crown elevation. Maximum lowerings of approximately one stone diameter occurred. An additional 30 min (prototype time) was added to Step 3 of Hydrograph A for this test and damage continued to occur throughout this extended time. Although the damage had not stabilized, the test was stopped. The amount of damage accrued by the revetment crown exceeded an acceptable amount for a stable design. Photographs 31-33 show the condition of Plan 5 at the end of this test. Since Plan 5 proved to be an unacceptable design for the wave and swl conditions of Hydrograph A, it was not tested for the wave and swl conditions of Hydrograph B.

Plan 6

29. Tests were conducted for Plan 6 (Plate 6 and Photos 34-36) to check the stability of 4,200-lb, special-placed armor stone when exposed to the wave and swl conditions of Hydrograph A. By the conclusion of Hydrograph A, the outer sea-side toe had accrued moderate to significant damage, the sea-side of the crown had sustained minor to moderate

damage, and the landside toe showed some minor damage (Photos 37-39). Nine stones were displaced off the outer sea-side toe and several other stones were pushed in a landward direction. Comparison of Photos 34 and 37 shows that stones on the sea side of the crown were displaced landward causing a shortening of the original 16-ft crown width. One armor stone was displaced out of its original position on the landside toe, but this did not appear to have any effect on the landside slope stability. Several stones on the sea-side toe and slope and on the revetment crown rocked in place throughout the test, but armor-stone displacement stopped during the early part of Step 3 of Hydrograph A.

30. Without rebuilding the test section, Plan 6 was exposed to Hydrograph B. These test conditions caused no additional damage.

High-water-level tests of Plan 4

31. Plan 4 (Plate 4) was selected for the prototype revetment if a gabion toe is used on the final design. Plan 4 was reconstructed in the test flume (Photos 40-42) and exposed to Hydrographs A and C. At the completion of Hydrograph A, all damage had stopped and six stones were off the outer sea-side toe (minor to moderate damage). No other damage occurred during Hydrograph A. The wave and swl conditions of Hydrograph C produced very severe overtopping conditions, but only caused a minor amount of additional revetment damage. During Step 5 of Hydrograph C, two armor stones from the revetment crown were displaced onto the landside of the structure. No other displacement occurred, but a larger number of stones on the upper sea-side slope and crown showed significant rocking in place throughout the high-water-level tests. All damage subsided by the end of the test and Photos 43-45 show the condition of Plan 4 after testing.

Plan 7

32. Following changes in the test flume bathymetry (Figure 5c), Plan 7 (Plate 7 and Photos 46-48) was constructed and exposed to Hydrographs A, B, and C in order to check the stability of the Sta-Pod toe configuration in concert with special-placed, 5,900-lb armor stone. The special-placed armor stone, underlayer stone, and bedding stone of Plan 7 were identical with Plan 4. The sea-side toe of Plan 7 was

approximately 14 ft wide and the outer sea-side toe was constructed of a single row of interlocked, 8,919-lb Sta-Pods. The 5,900-lb, special-placed armor stone accrued only slight damage during exposure to Hydrograph A. One armor stone was displaced from the center of the sea-side slope down onto the armor-stone toe. Several armor stones on the sea-side toe, sea-side slope, and crown displayed minor to moderate rocking in place throughout Hydrograph A, but no additional armor-stone displacement occurred. Photos 49-51 show the condition of Plan 7 at the end of Hydrograph A.

33. Following exposure to Hydrograph A, Plan 7 was subjected to the wave and swl conditions of Hydrograph C. The revetment sustained only minor additional damage during exposure to these test conditions. One armor stone was displaced from the landside of the crown down to the toe of the landside slope, and some loosening and reorientation of other crown stone occurred. Armor stone on the sea-side toe, sea-side slope, and crown showed significant rocking in place throughout the test, but no additional armor-stone displacement occurred. All damage had stopped and the structure was in good condition at the end of Hydrograph C (Photos 52-54).

34. Without rebuilding the test section, Plan 7 was exposed to Hydrograph B. These wave and swl conditions caused no additional damage, but some minor rocking in place of three or four armor stones occurred on the sea-side toe throughout the test.

Discussion

35. The 2-D stability tests have only addressed the revetment stability for wave attack where the incident wave direction is perpendicular to the revetment crown. Previous model and prototype experience has shown that this is the worst wave condition that can occur in regard to runup and stability on a continuous length of armor-stone breakwater trunk or revetment. This is not necessarily the worst incident wave angle where discontinuities occur, such as the ends of the revetment protection or where the revetment bends along an existing irregular

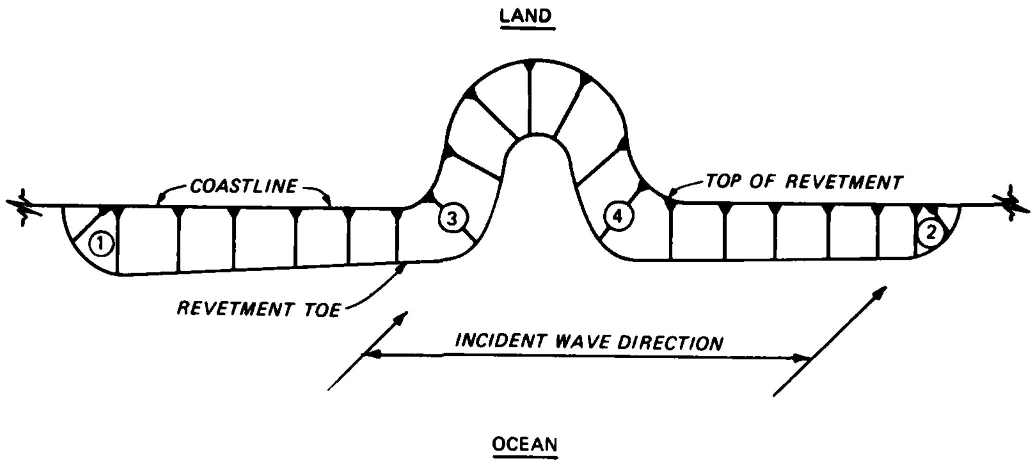


Figure 6. Areas of lower armor-stone stability. Areas (1) and (2) are ends of revetment. Areas (3) and (4) are discontinuities caused by revetment bending along an irregularity in the coastline

coastline (Figure 6). These conditions are somewhat analogous to a breakwater head. Extensive model tests have shown that breakwater heads exhibit lower stabilities than breakwater trunks when exposed to the same wave conditions.

36. Due to the state of the art of movable-bed modeling and the limitations of the 2-D model (no longshore currents or angular wave attack), the problem of possible scour along the revetment toe could not be addressed. The width of the gabion toe on Plan 4 should be sufficient if only minor scour and/or undermining of the toe occurs. Minor amounts of undermining and toe scour on Plan 7 could cause the Sta-Pods to settle and/or overturn. If this should occur, the buttressing of the toe armor stone provided by the Sta-Pods would be lost. This could result in failure of the armor-stone toe which could initiate a slide-type failure of the sea-side slope armor stone. For this reason, it is felt that the bedding layer under the Sta-Pod toe on Plan 7 does not provide a sufficient width of toe protection; and it is suggested that the bedding stone and underlayer stone be extended at least 5 ft seaward of the Sta-Pods to provide some added stability against toe scour and undermining of the Sta-Pod toe.

37. The prototype gabions will most likely have a rough texture

and due to differential settlement, the elevation of the gabion tops will vary. The model gabions were constructed out of window screen and had a very smooth, uniform surface. For this reason, the outer toe stones in the prototype may show a somewhat higher stability than those observed in the model. On the other hand, if the outer prototype gabion toe settles due to undermining, this could create a gabion top surface that slopes away from the structure. If this occurs, the outer toe stones may have a lower stability than what was observed on the model.

PART IV: CONCLUSIONS

38. Based on the test conditions and test results reported herein, it is concluded that:

a. Plan 1 (special-placed, 8,600-lb armor stone) is an adequate design, provided a minor amount of sea-side toe damage is acceptable.

b. Plan 2 (random-placed, 8,600-lb armor stone) is an adequate design, provided care is taken to obtain a good crown construction and a minor amount of sea-side toe damage is acceptable.

c. Plan 3 (random-placed, 5,900-lb armor stone) is a marginally acceptable design, exhibiting minor to moderate sea-side toe damage and moderate sea-side slope and crown damage.

d. Plan 4 (special-placed, 5,900-lb armor stone) is an adequate design, provided a minor amount of sea-side toe damage is acceptable.

e. Plan 5 (random-placed, 4,200-lb armor stone) is not an adequate design.

f. Plan 6 (special-placed, 4,200-lb armor stone) is not an adequate design for the sea-side toe, but is a marginally acceptable design for the remainder of the revetment.

g. Plan 7 (special-placed, 5,900-lb armor stone with an 8,919-lb, Sta-Pod toe) is an adequate design with the possible exception of toe protection (see recommendations).

h. Plans 4 and 7 will accrue only a minor amount of additional damage when exposed to Hydrograph C (wave and swl conditions that exceed the design conditions).

PART V: RECOMMENDATIONS

39. The adequacy of the Plan 2 and the borderline acceptability of the Plan 3 design are highly dependent upon obtaining a good prototype crown construction. If a totally random construction technique is used on the crown, it is highly probable that both designs could fail if they are exposed to waves similar in magnitude to those of Hydrograph A. Thus, it is recommended that care be taken to ensure that a good prototype crown construction is achieved.

40. It is recommended that the bedding layer and underlayer stone be extended at least 5 ft seaward of the Sta-Pods of Plan 7. This should provide some added stability against toe scour and undermining of the Sta-Pod toe.

41. Based on the points discussed in paragraph 35, it is recommended that the larger sizes of the selected armor-stone protection (top 25 percent by weight) be placed on the revetment ends to provide added buttressing in these areas. These larger stones should also be used to provide added stability at points of irregularity along the revetment length.

Table 1

Hydrographs A, B, and C

Step	Still-Water Level ft NGVD	Test Wave Period sec	Test Wave Height ft	Prototype Duration hr	Wave Type
		Hydrograph A			
	+10.7	8	5.5	0.25	Shakedown
1	+10.7	8	10.1	1.0	Worst breaking
2	+10.7	10	11.8	1.0	Worst breaking
3	+10.7	12	10.6	1.0	Worst breaking
		Hydrograph B			
1	+ 8.0	8	8.1	*	Worst breaking
2	+ 8.0	10	8.5	*	Worst breaking
3	+ 8.0	12	11.7	*	Worst breaking
4	+ 5.0	8	5.5	*	Worst breaking
5	+ 5.0	10	6.4	*	Worst breaking
6	+ 5.0	12	6.9	*	Worst breaking
		Hydrograph C			
1	+13.0	8	11.0	0.5	Worst breaking
2	+13.0	10	11.0	0.5	Worst breaking
3	+13.0	12	16.3	0.5	Worst breaking
4	+15.0	8	11.6	0.5	Worst breaking
5	+15.0	10	13.9	0.5	Worst breaking
6	+15.0	12	17.2	0.5	Worst breaking

* There was no fixed test duration. Each step was run for a sufficient period of time to be sure that either no damage was going to occur, or that no additional damage would occur.

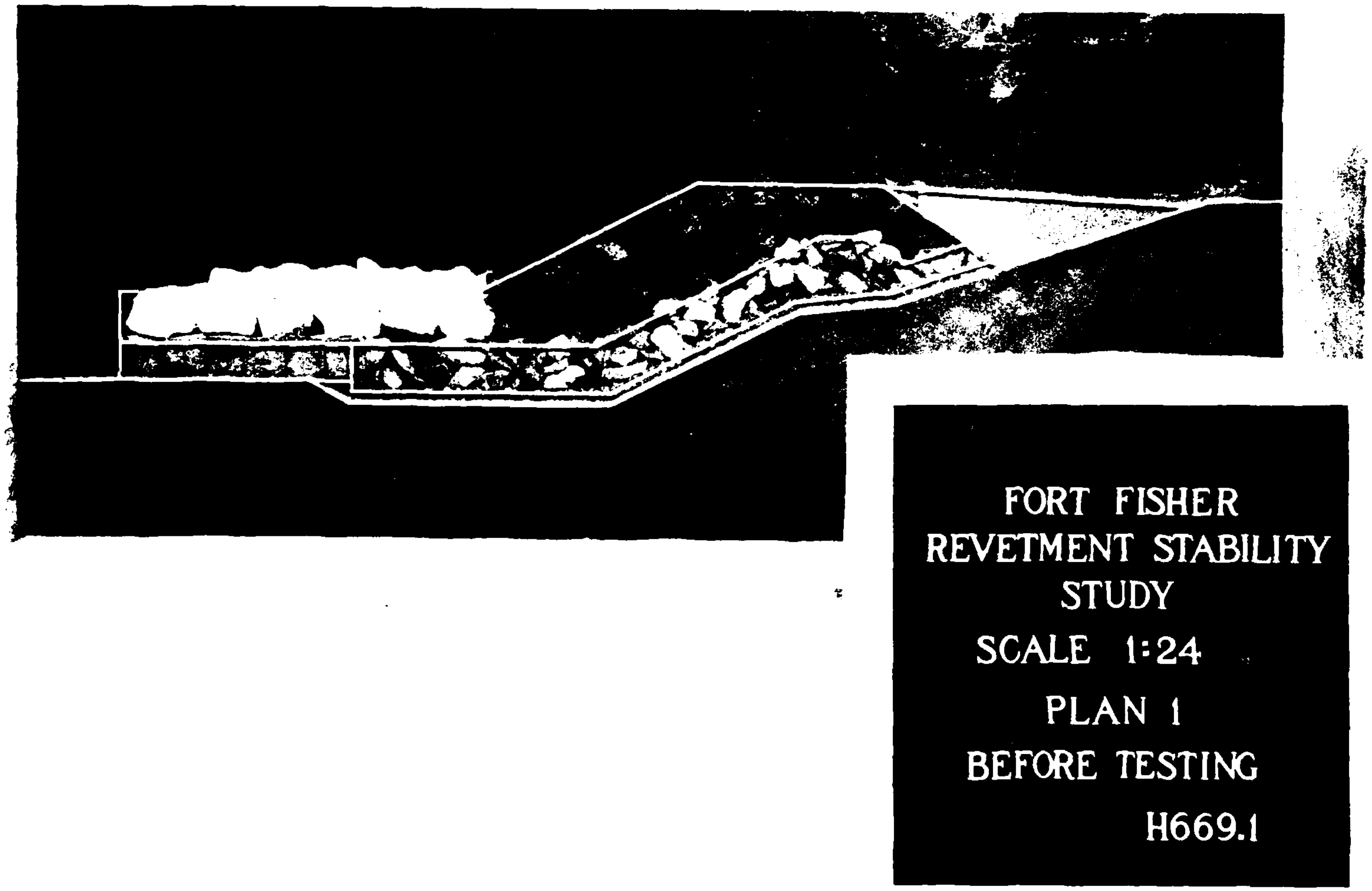


Photo 1. Side view of Plan 1 before testing

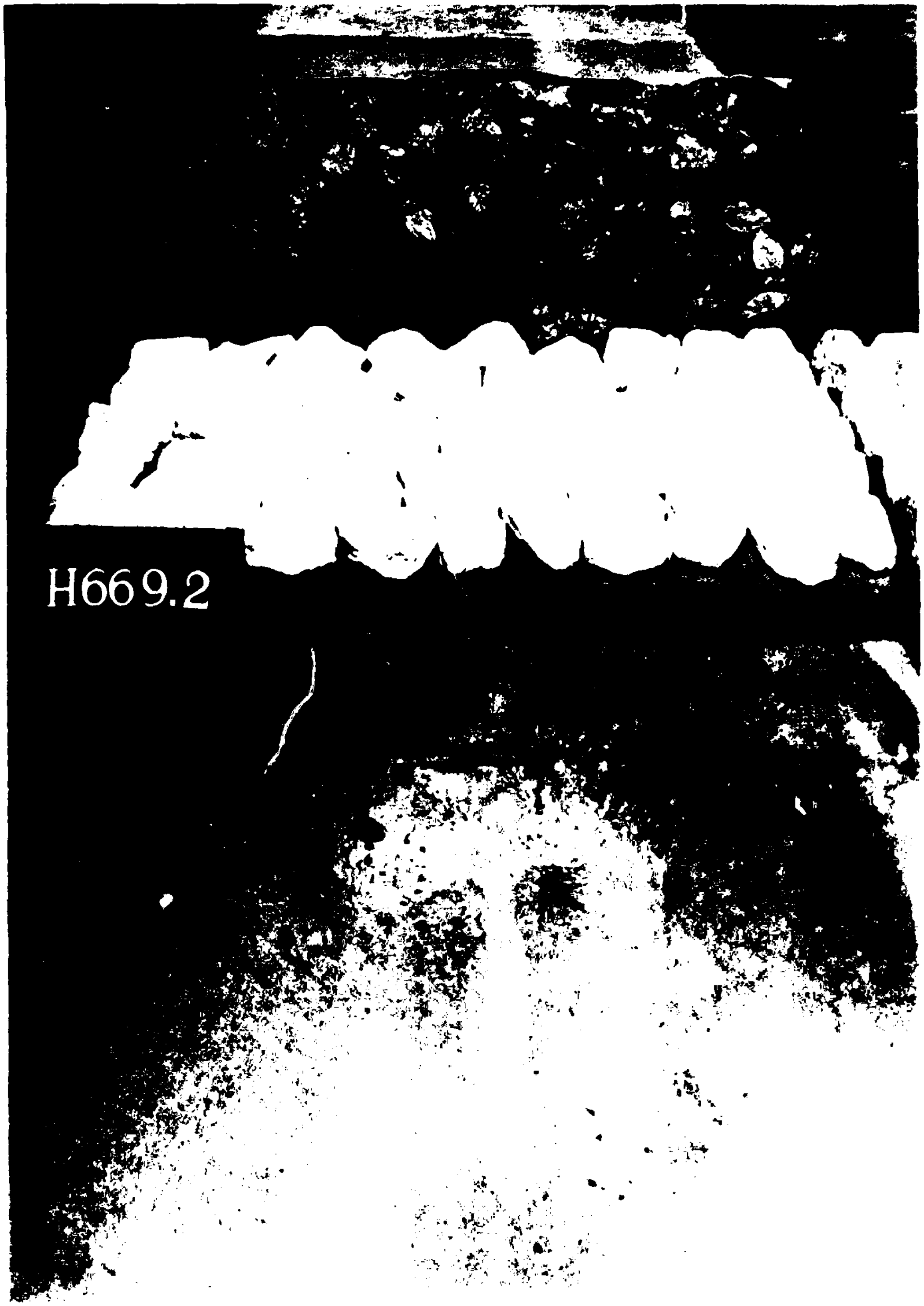


Photo 2. Sea-side view of Plan 1 before testing

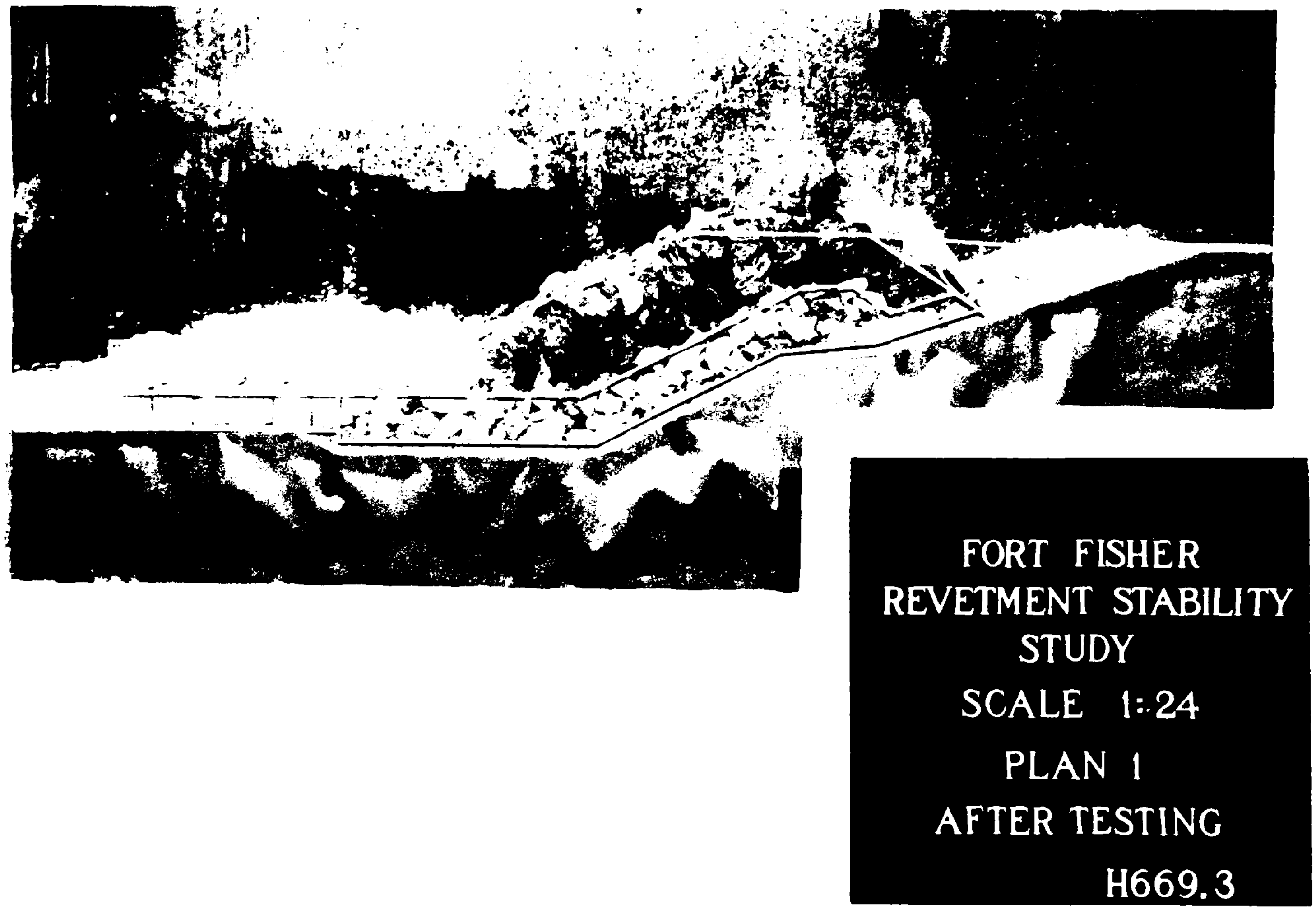


Photo 3. Side view of Plan 1 after testing

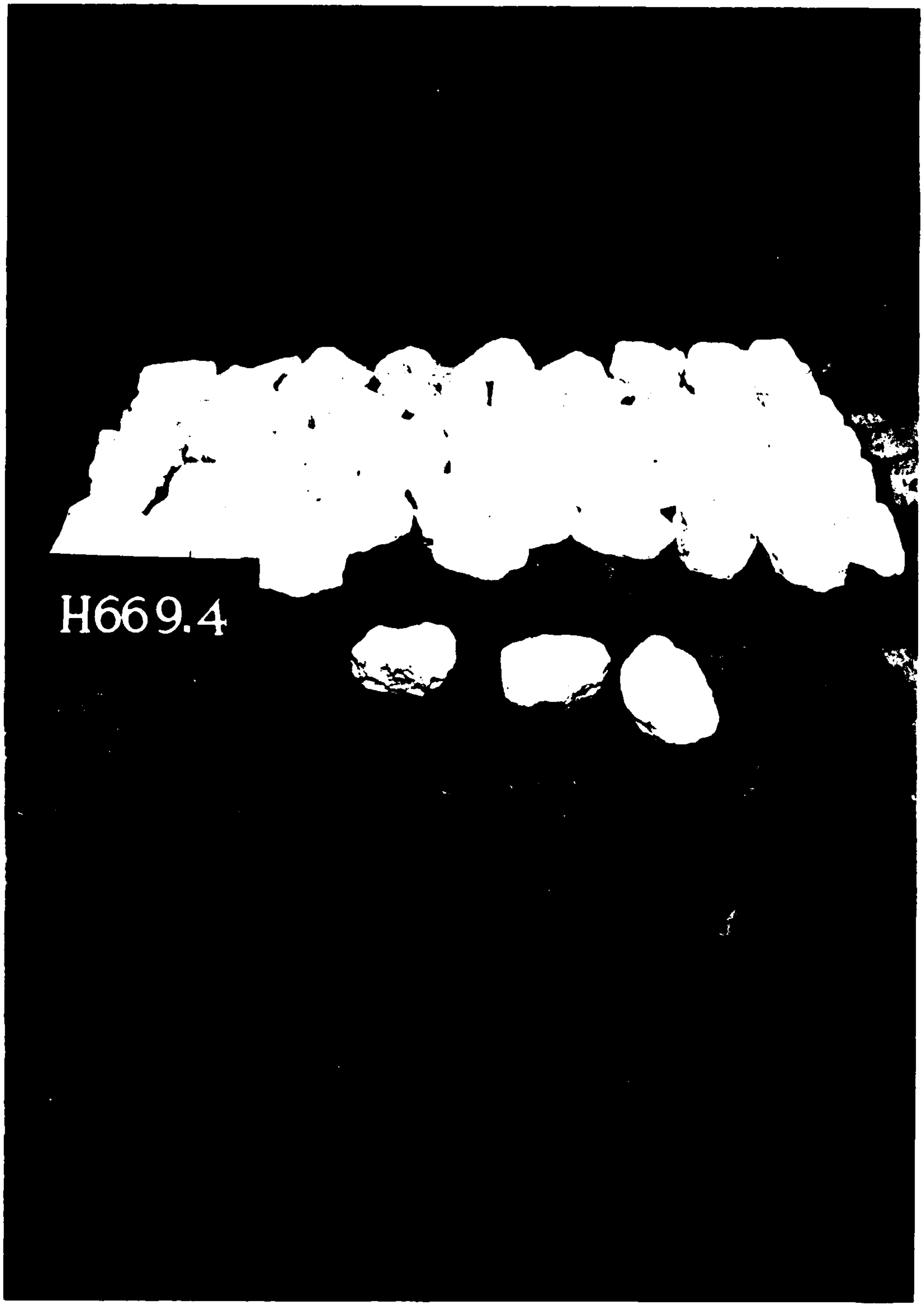


Photo 4. Sea-side view of Plan 1 after testing

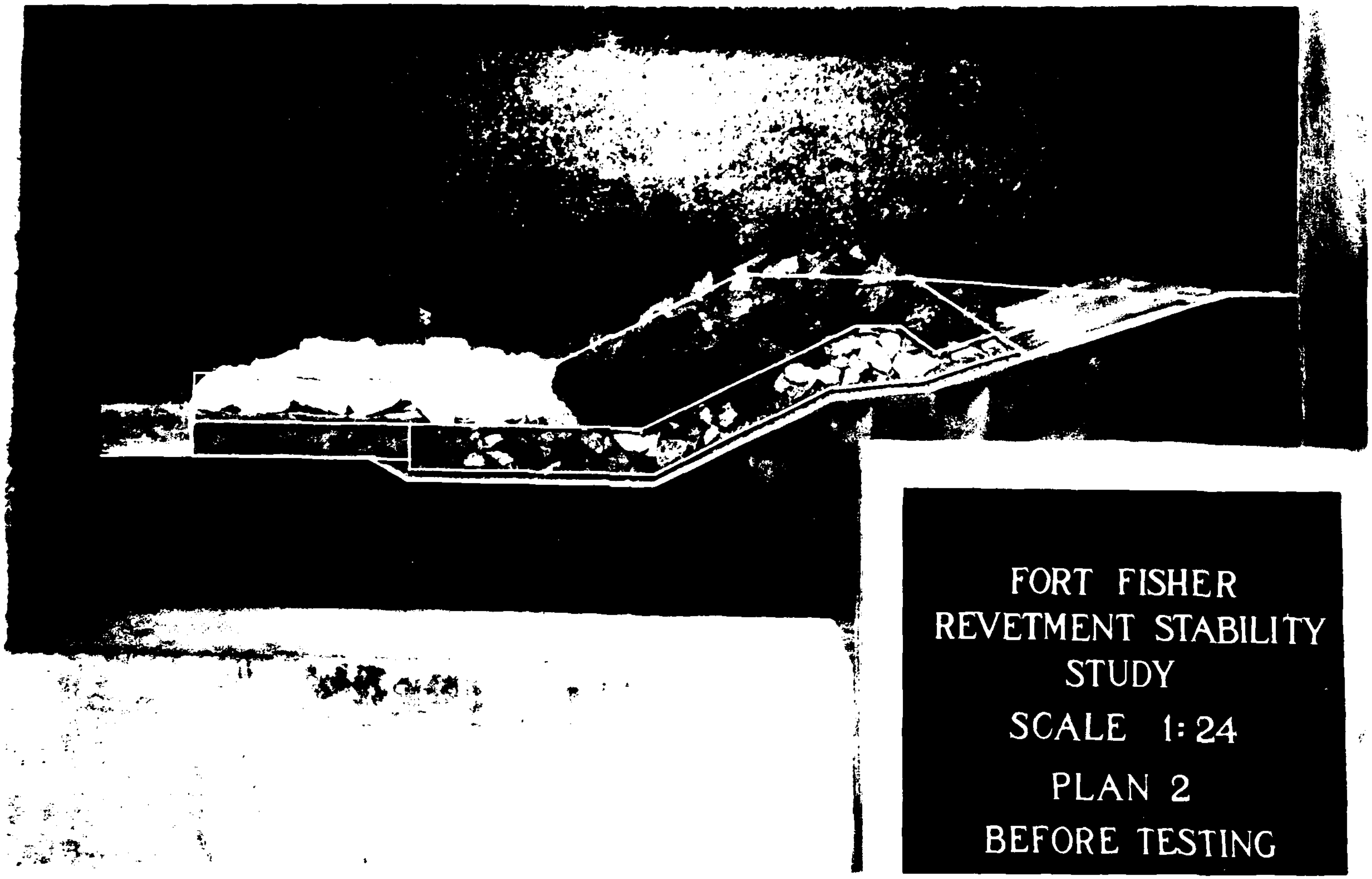


Photo 5. Side view of Plan 2 before testing, 1st test

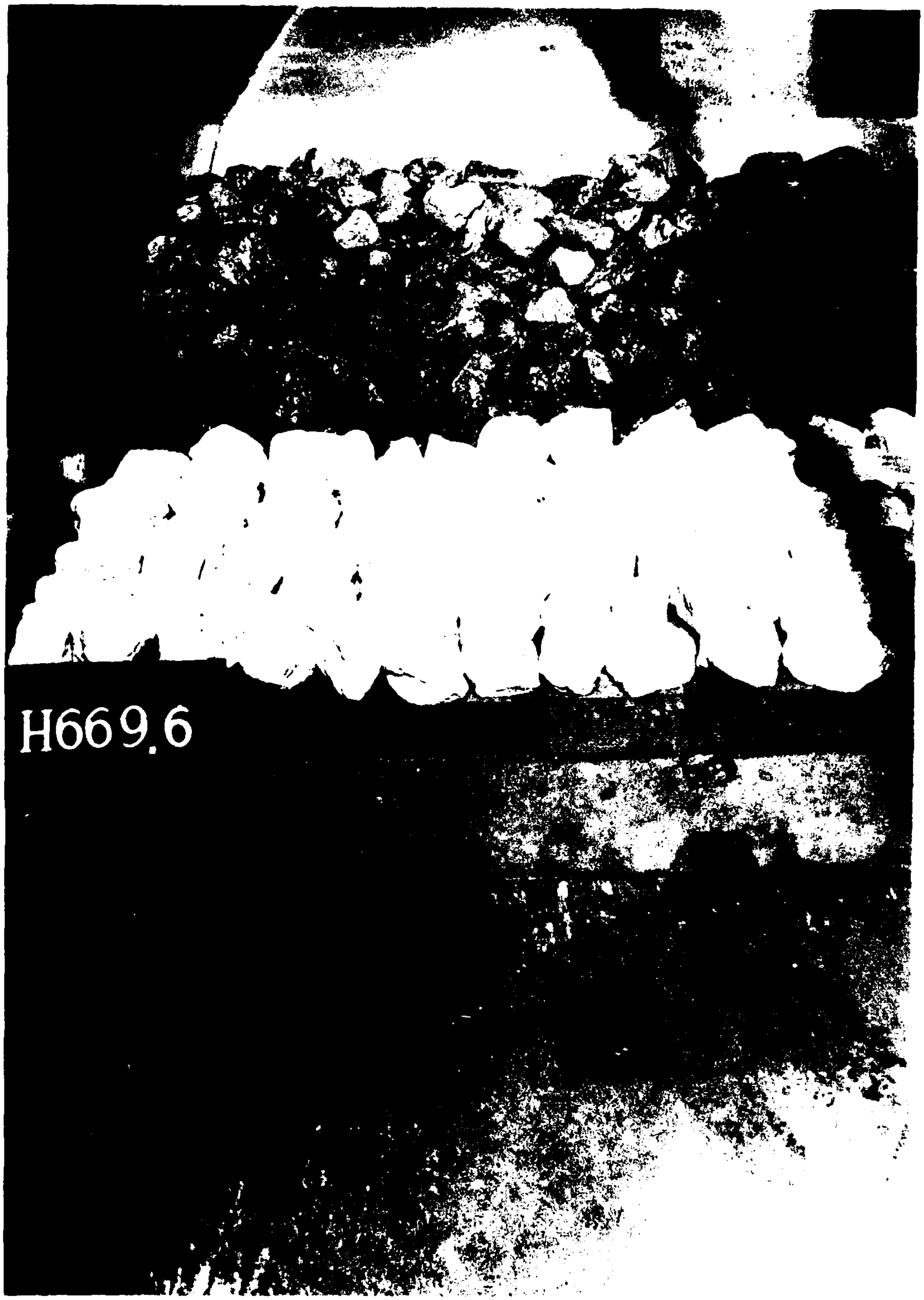


Photo 6. Sea-side view of Plan 2 before testing, 1st test

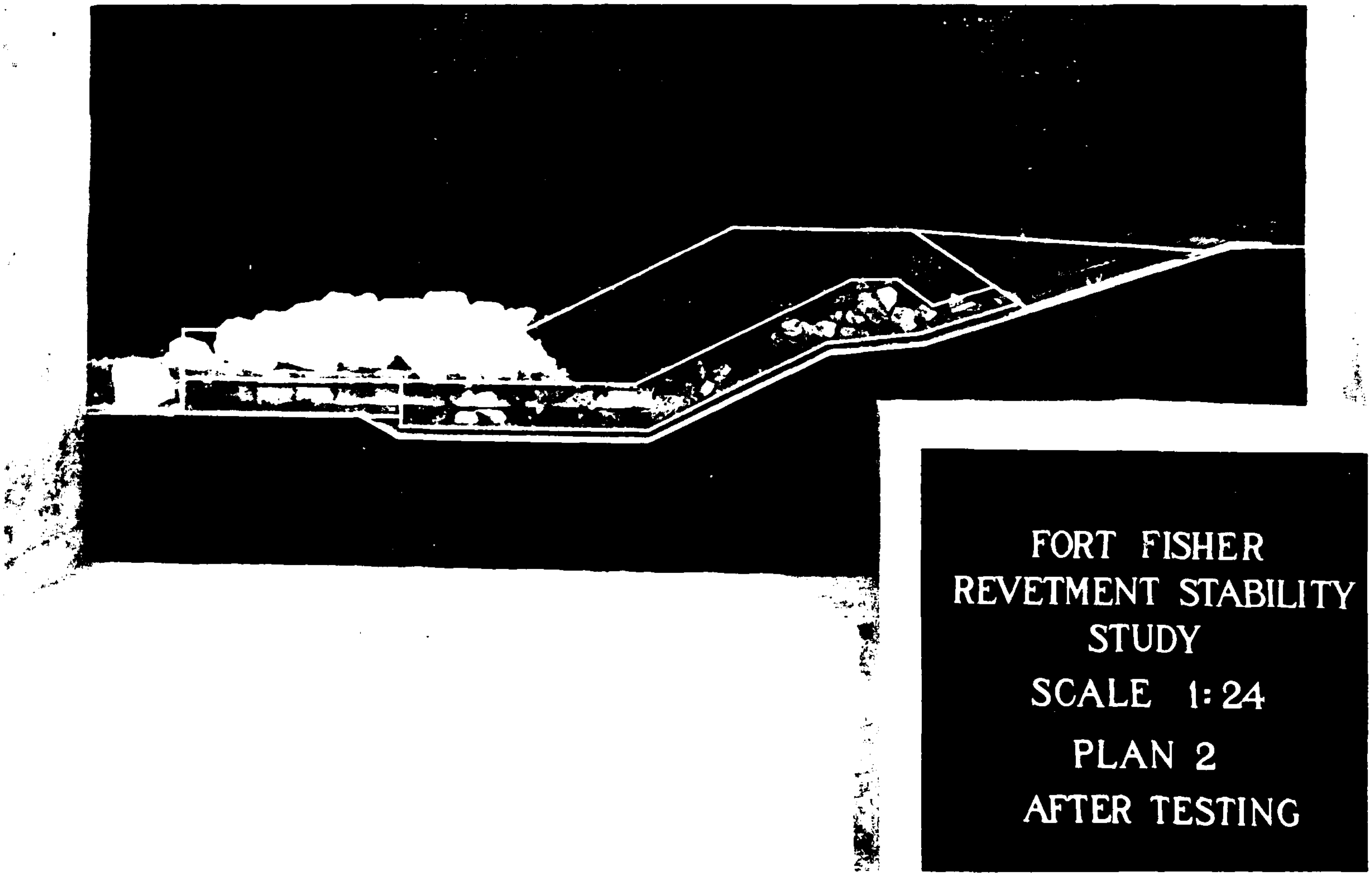


Photo 7. Side view of Plan 2 after testing, 1st test

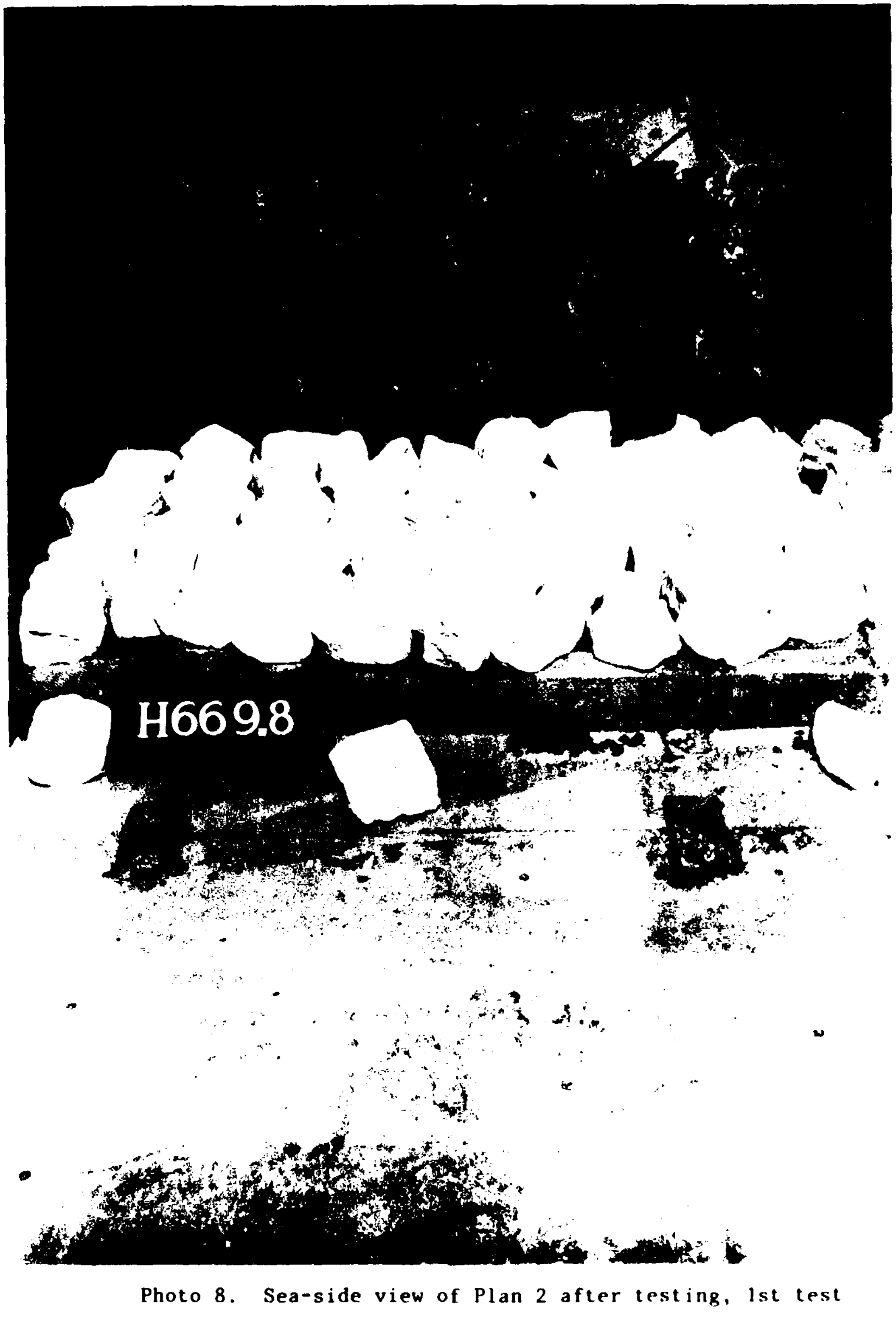


Photo 8. Sea-side view of Plan 2 after testing, 1st test

Photo 9. Landside view of Plan 2 after testing, 1st test

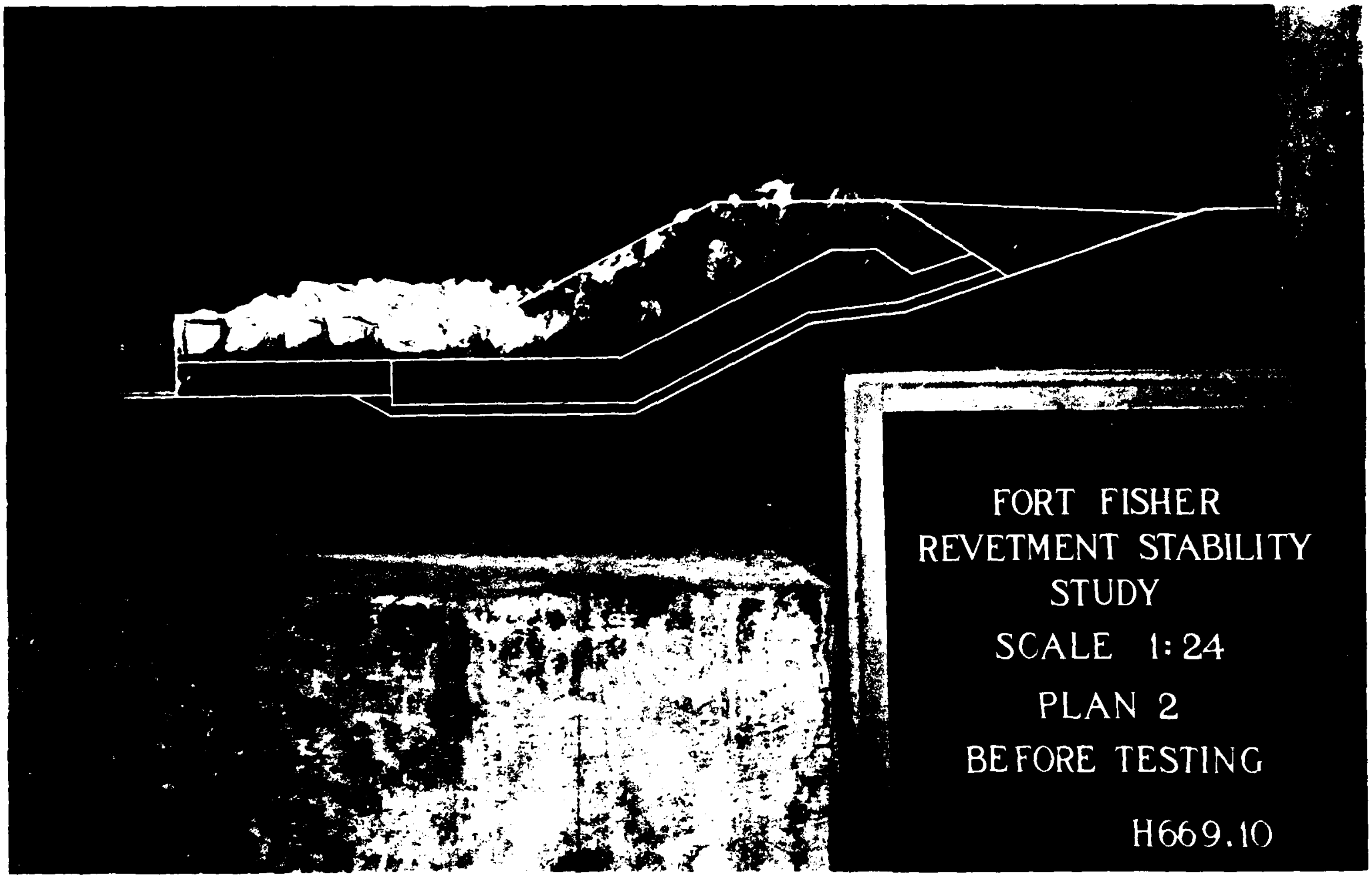


Photo 10. Side view of Plan 2 before testing, 2nd test

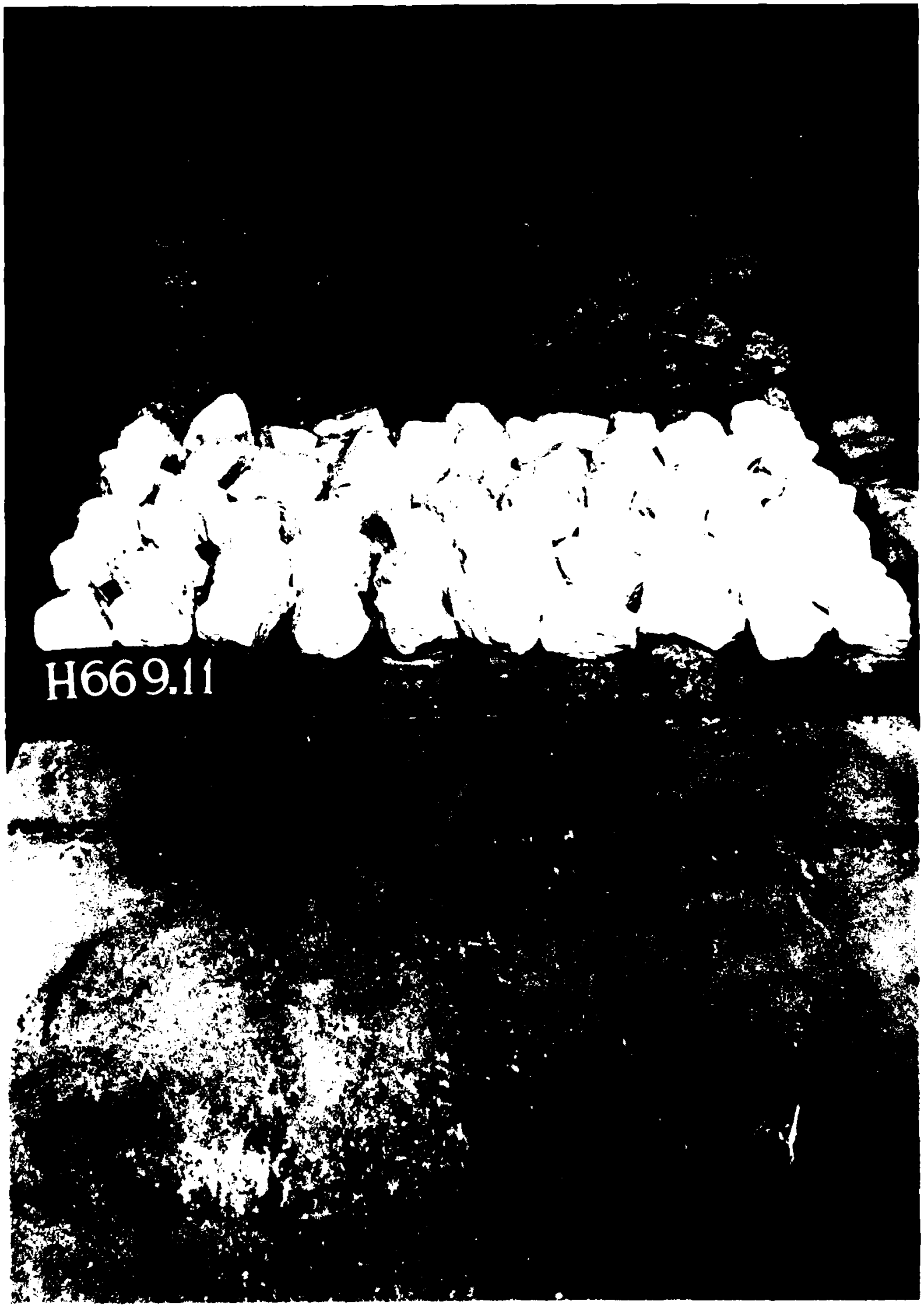


Photo 11. Sea-side view of Plan 2 before testing, 2nd test

Photo 12. Landside view of Plan 2 before testing, 2nd test

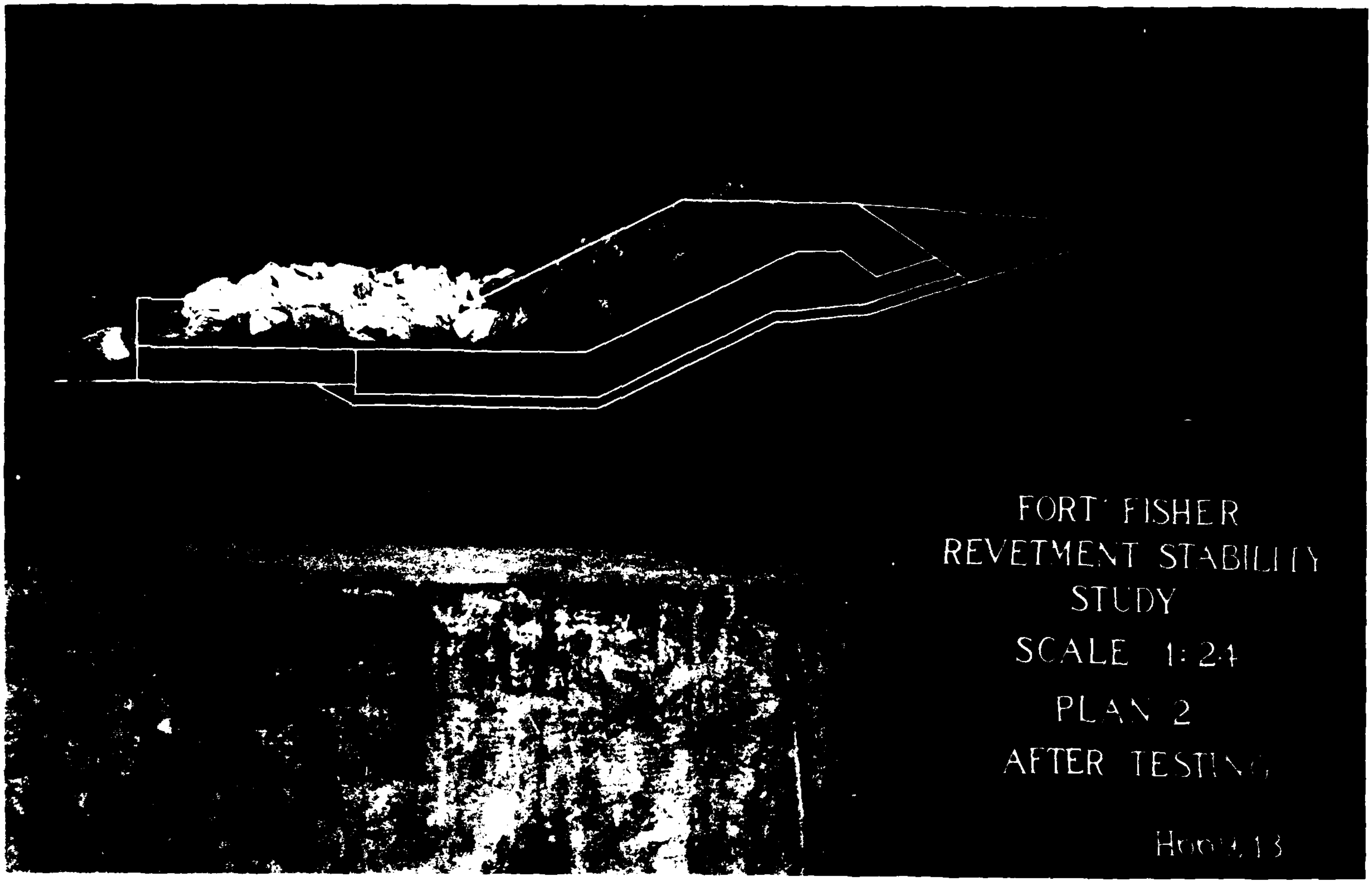


Photo 13. Side view of Plan 2 after testing, 2nd test

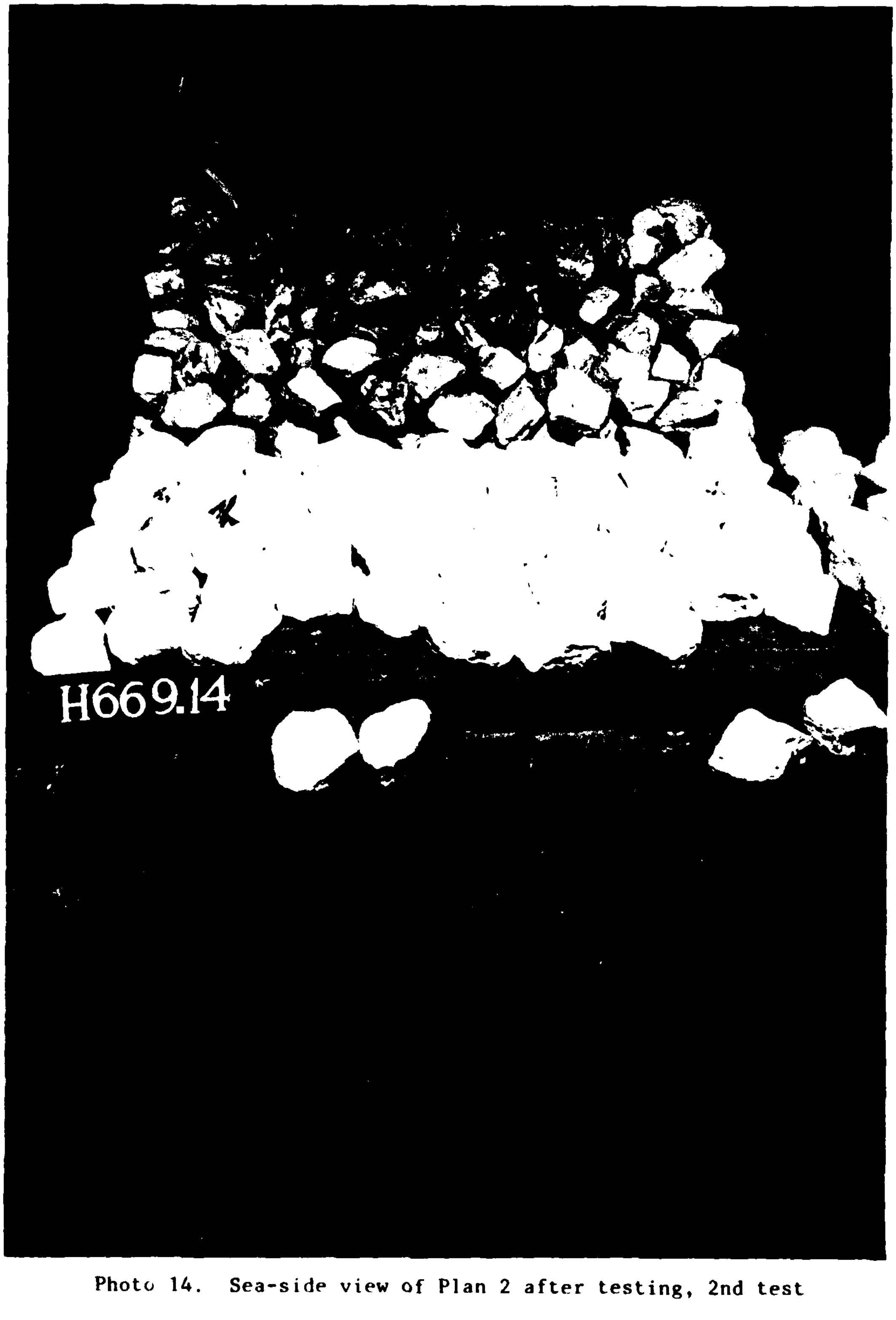


Photo 14. Sea-side view of Plan 2 after testing, 2nd test

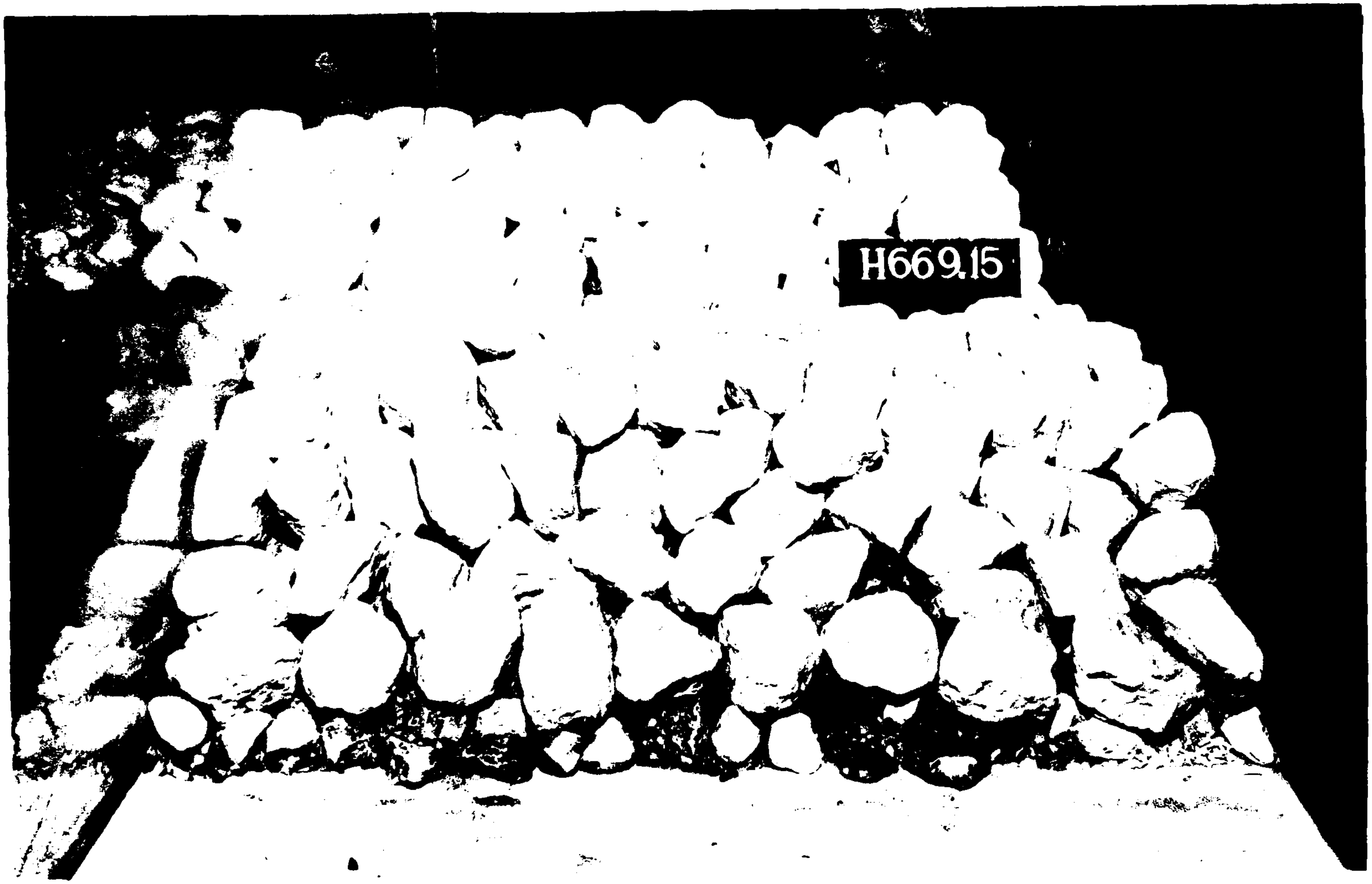


Photo 15. Landside view of Plan 2 after testing, 2nd test

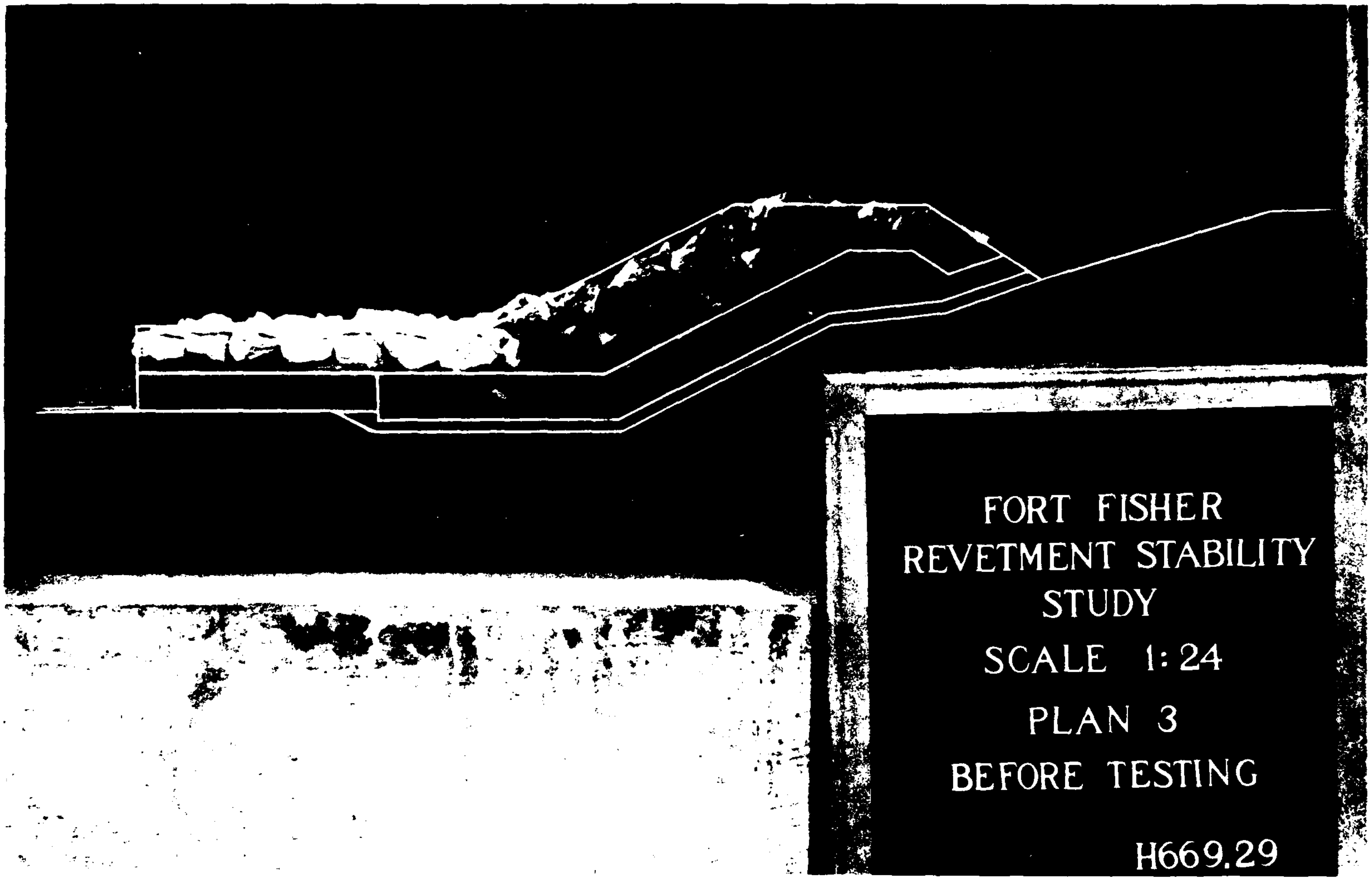


Photo 16. Side view of Plan 3 before testing

Photo 17. Sea-side view of Plan 3 before testing

Photo 18. Landside view of Plan 3 before testing

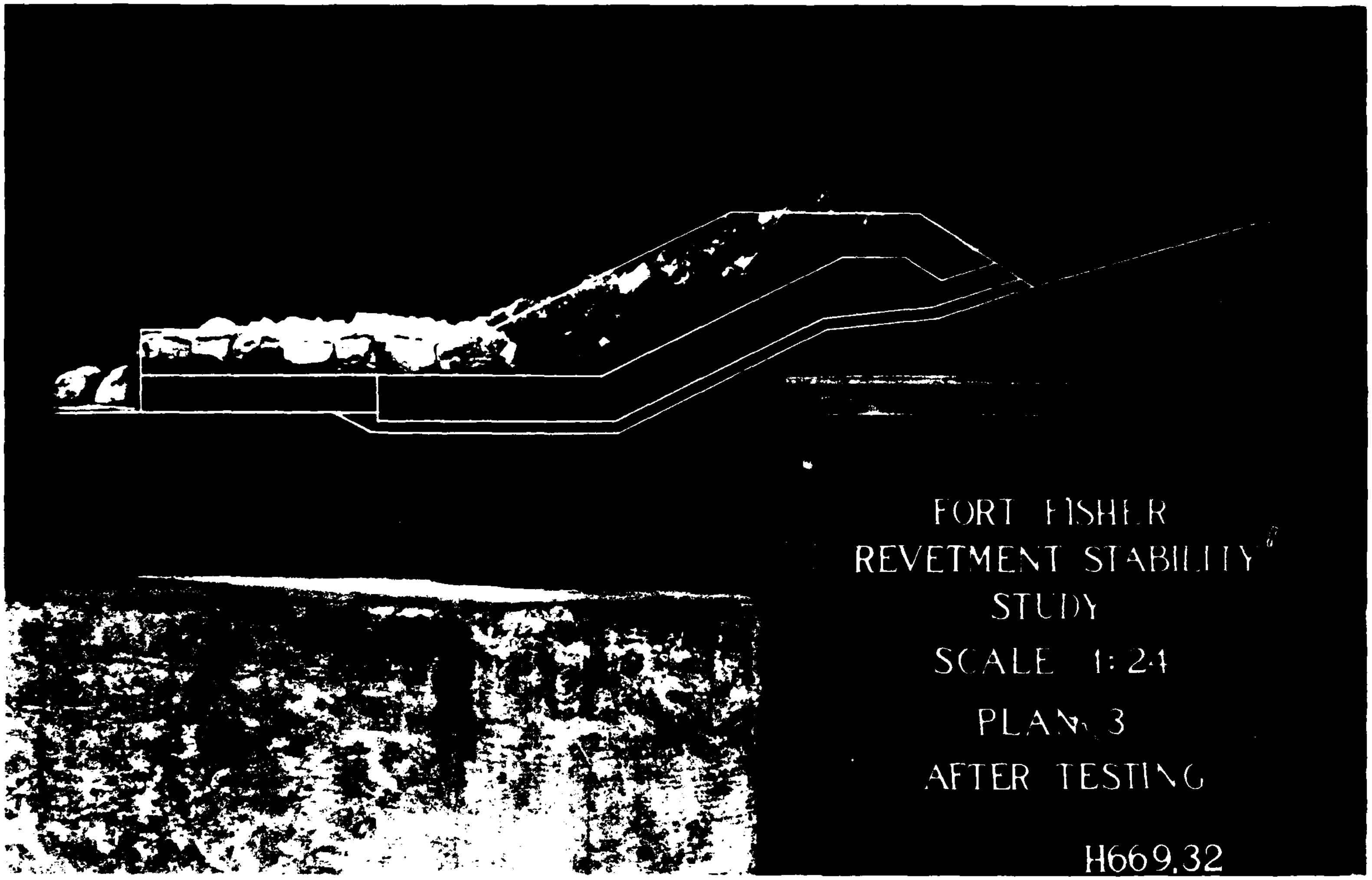


Photo 19. Side view of Plan 3 after testing

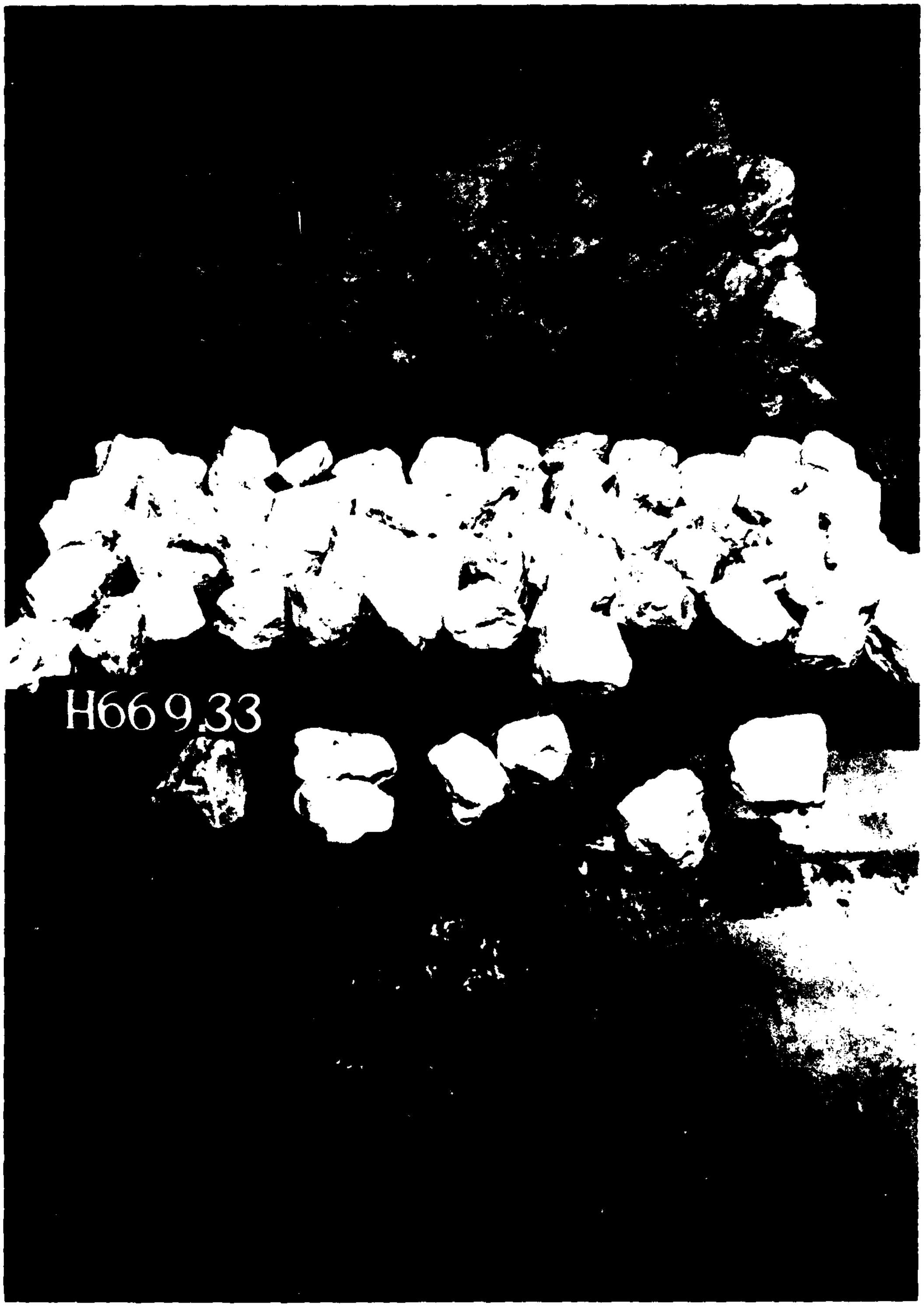


Photo 20. Sea-side view of Plan 3 after testing

Photo 21. Landside view of Plan 3 after testing

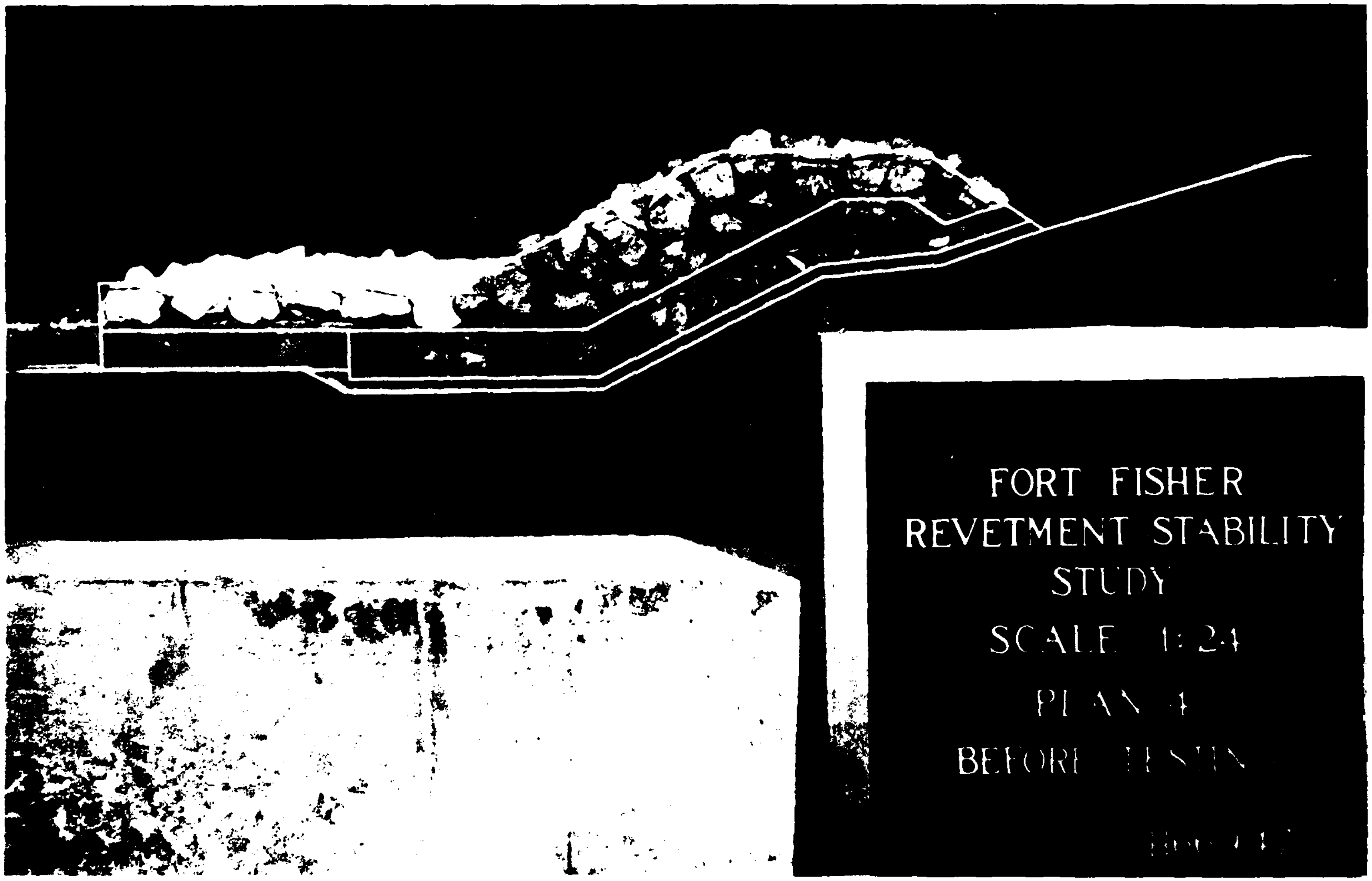


Photo 22. Side view of Plan 4 before testing

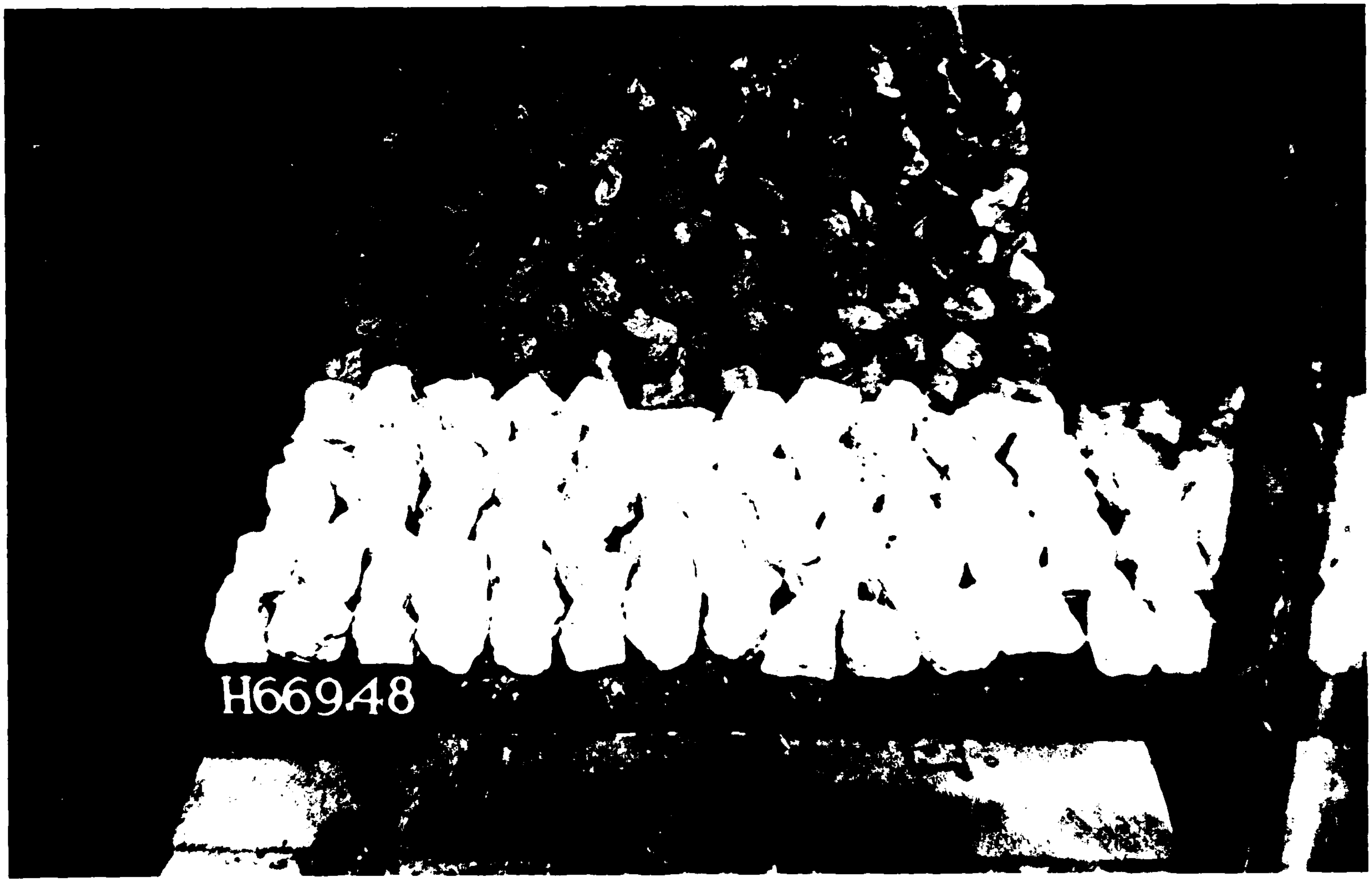


Photo 23. Sea-side view of Plan 4 before testing

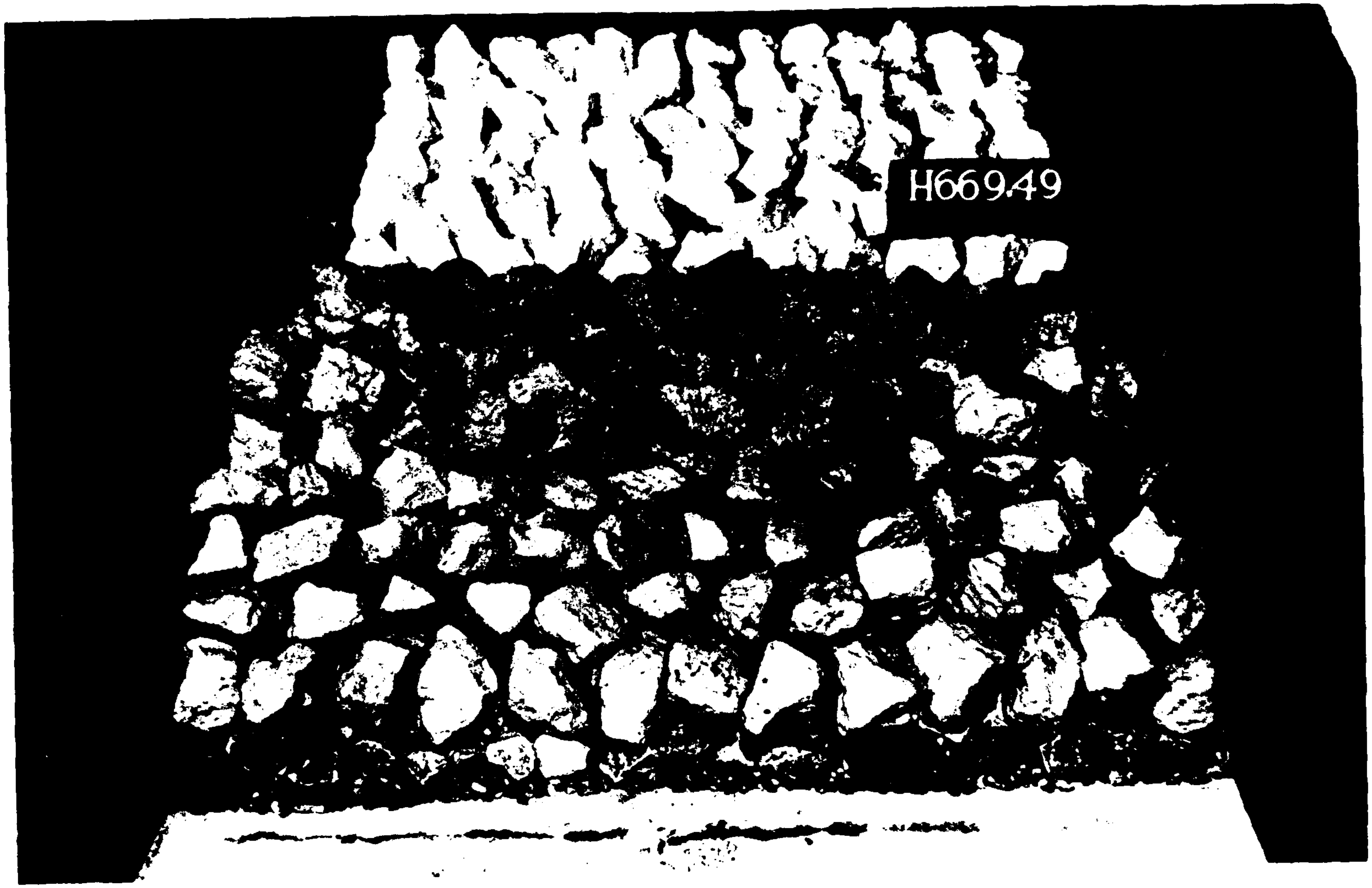


Photo 24. Landside view of Plan 4 before testing

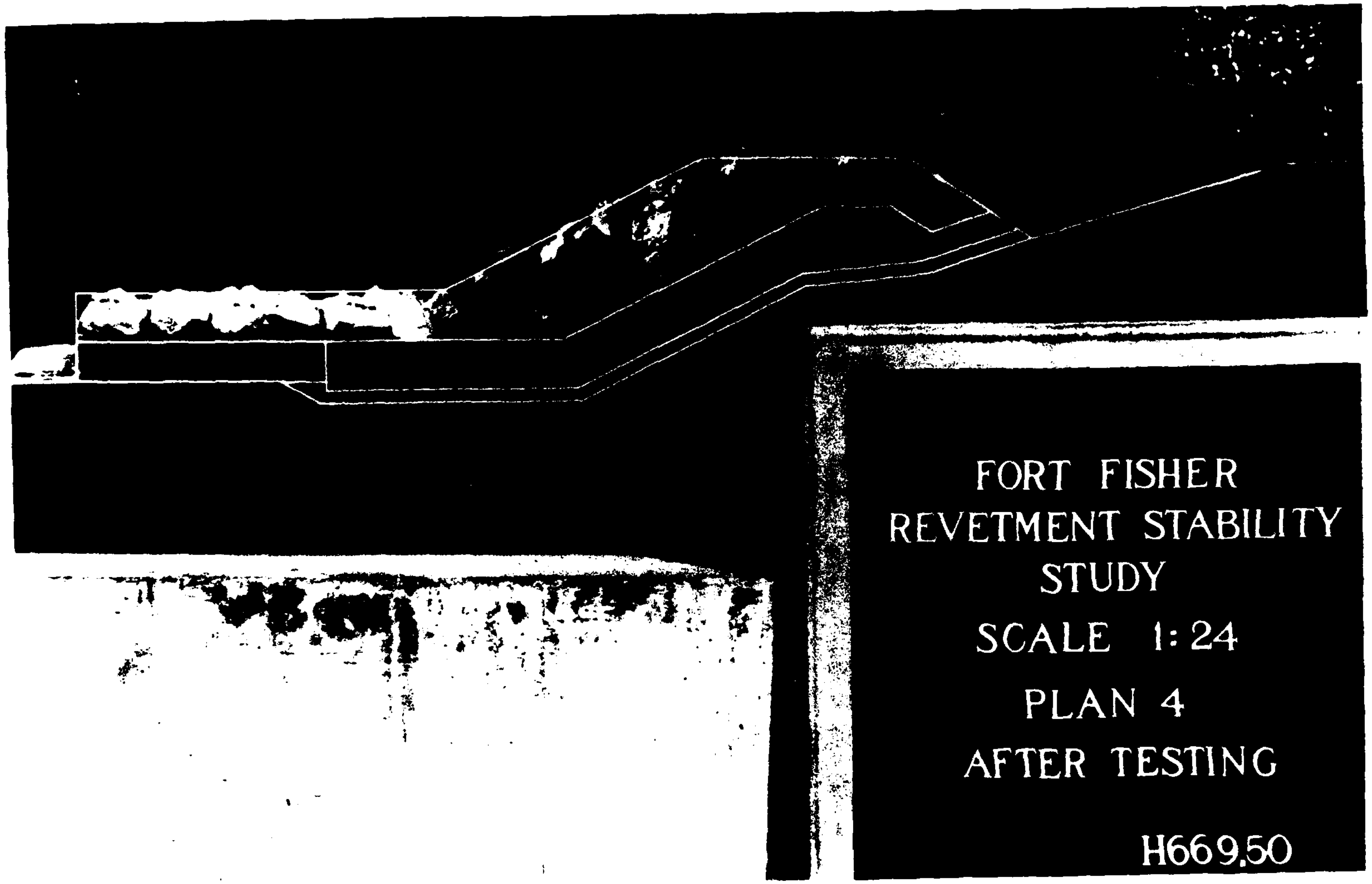


Photo 25. Side view of Plan 4 after testing

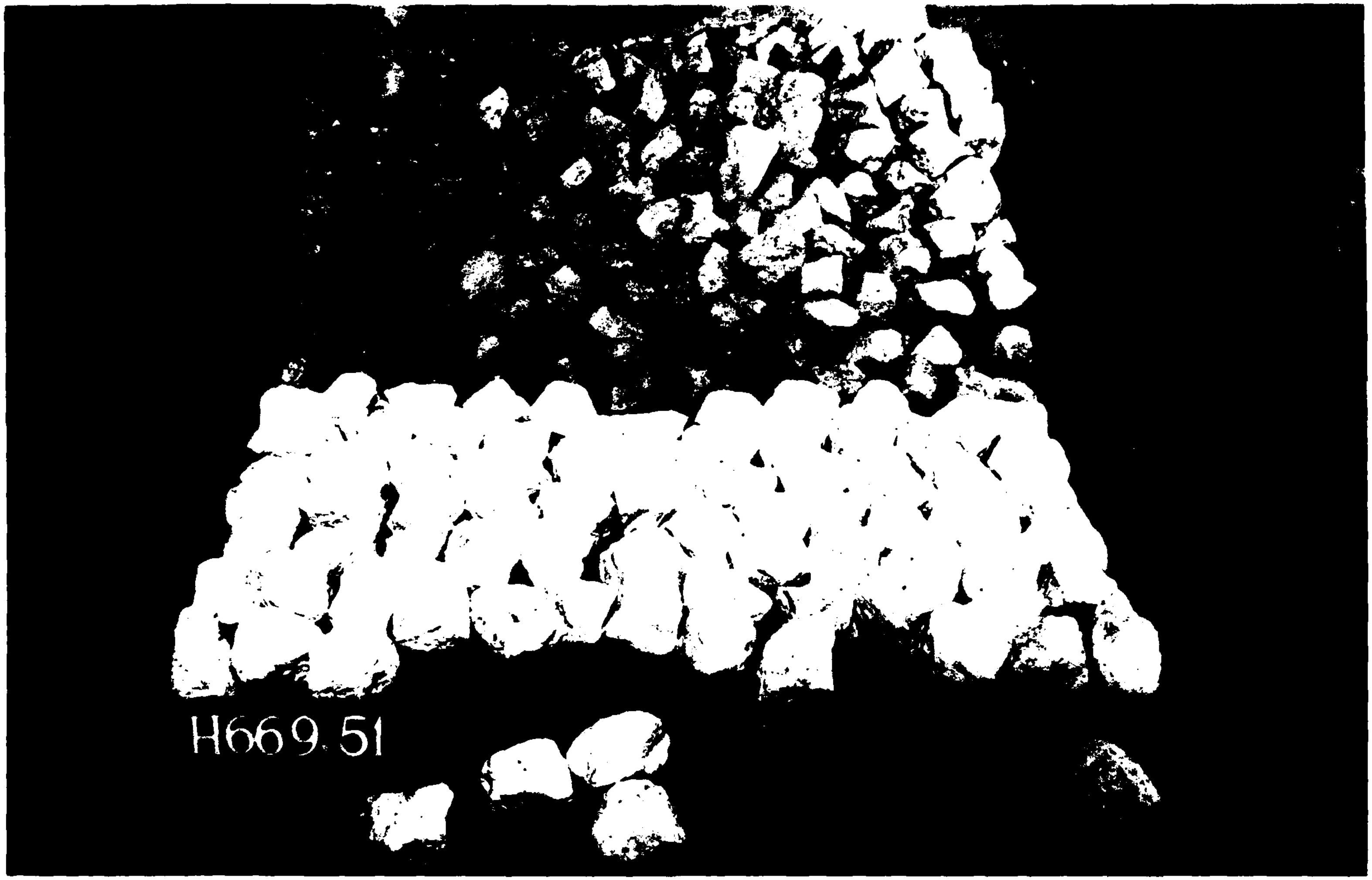


Photo 26. Sea-side view of Plan 4 after testing

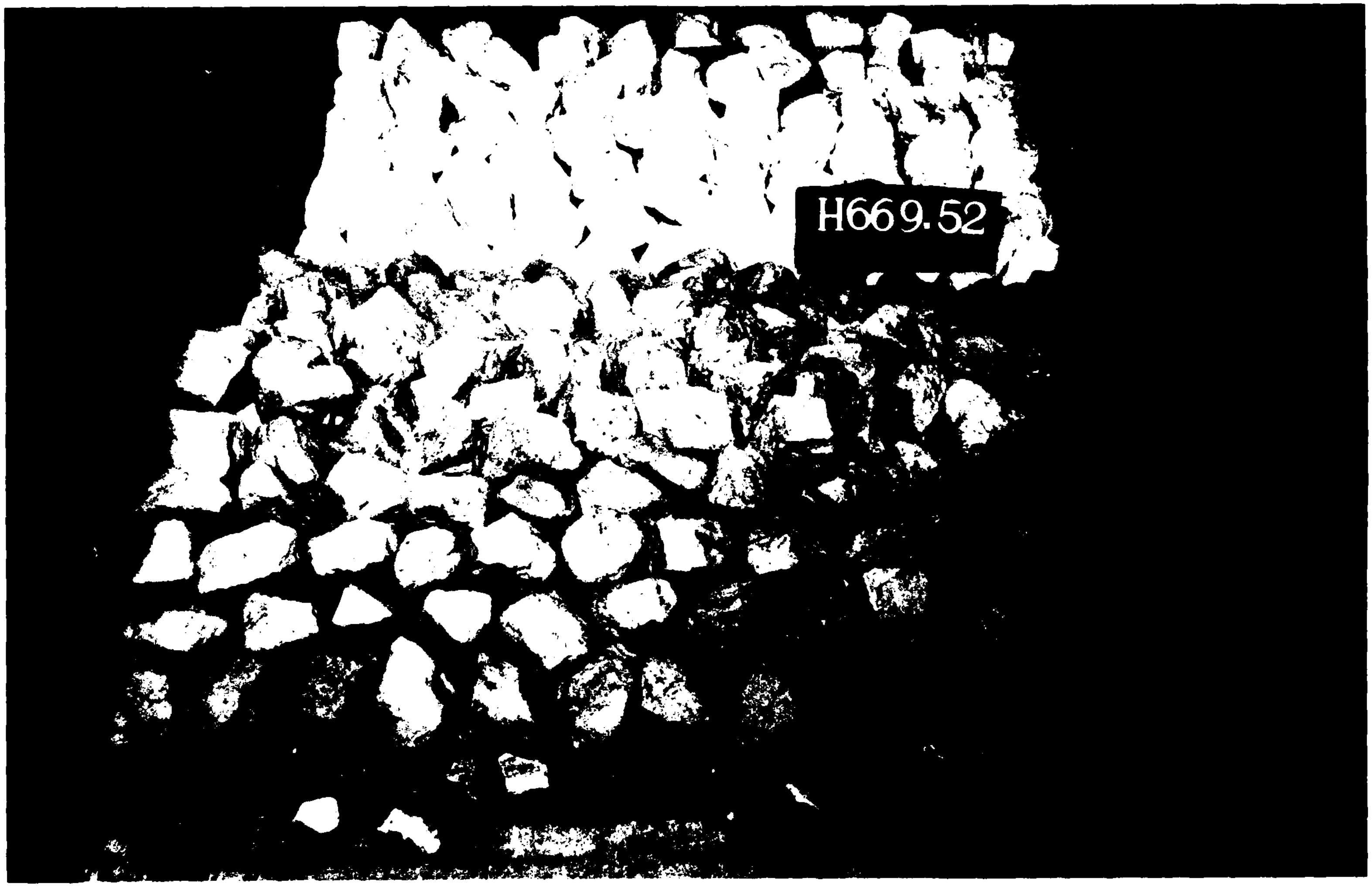


Photo 27. Landside view of Plan 4 after testing

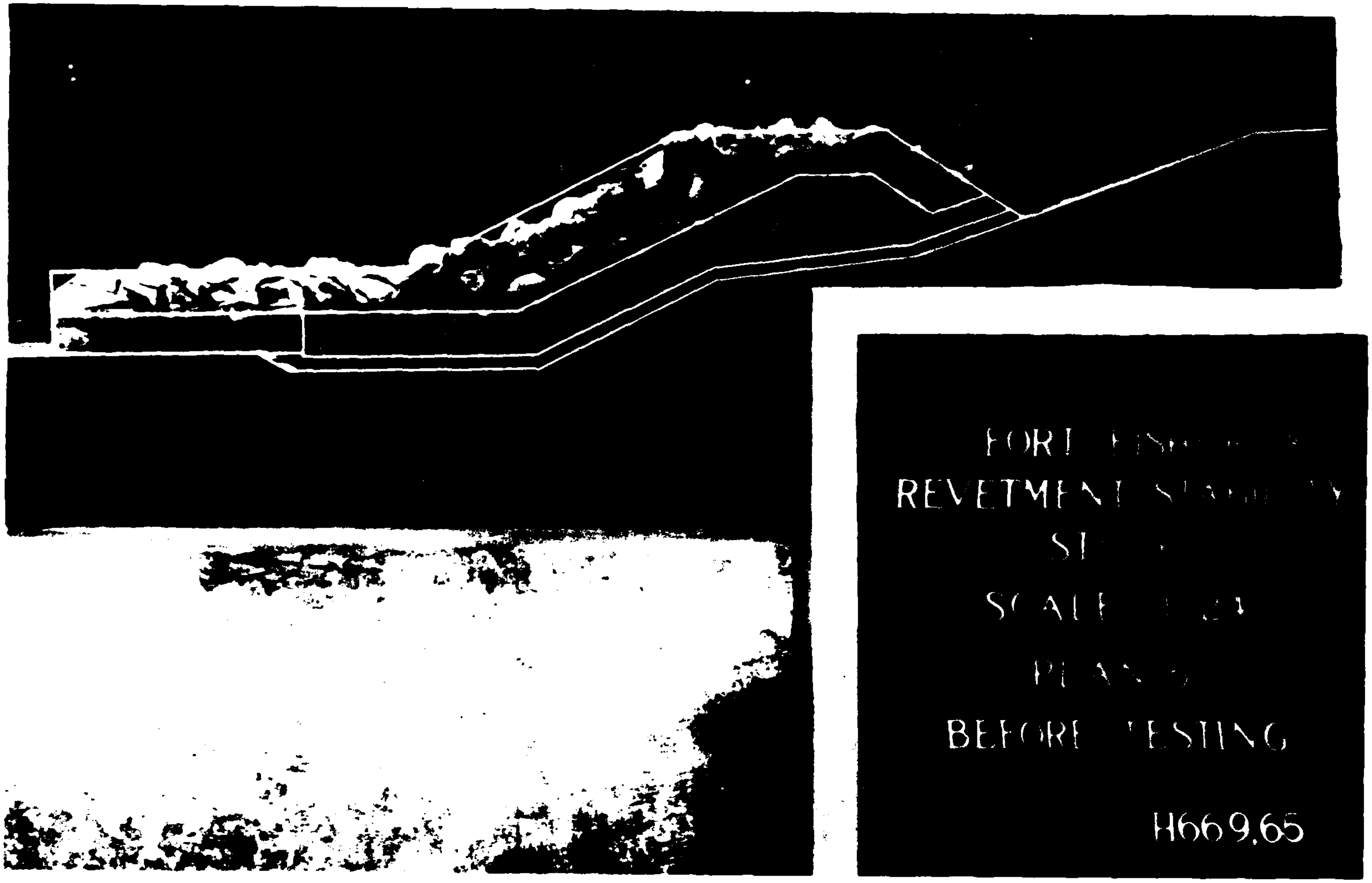


Photo 28. Side view of Plan 5 before testing

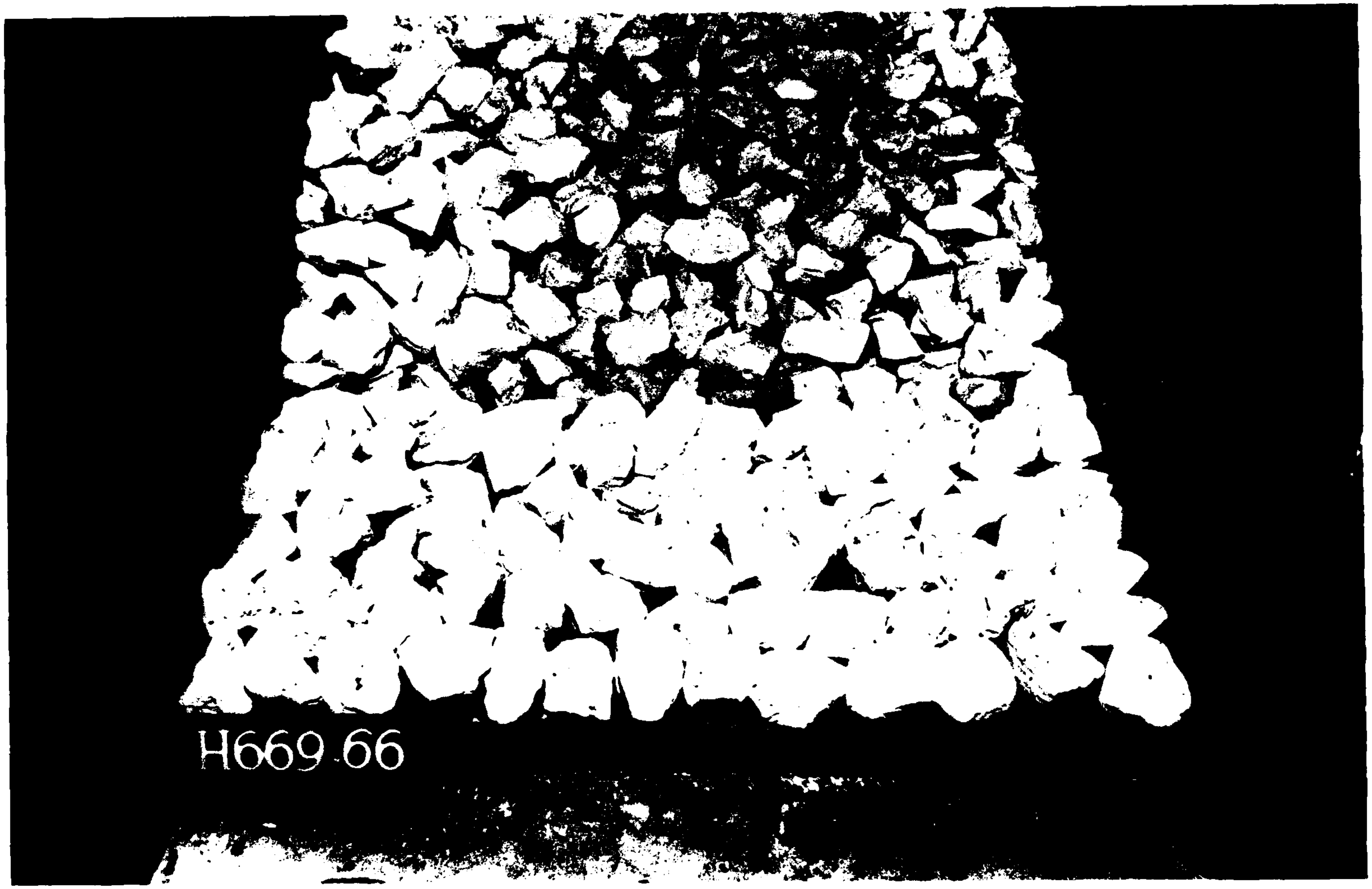


Photo 29. Sea-side view of Plan 5 before testing

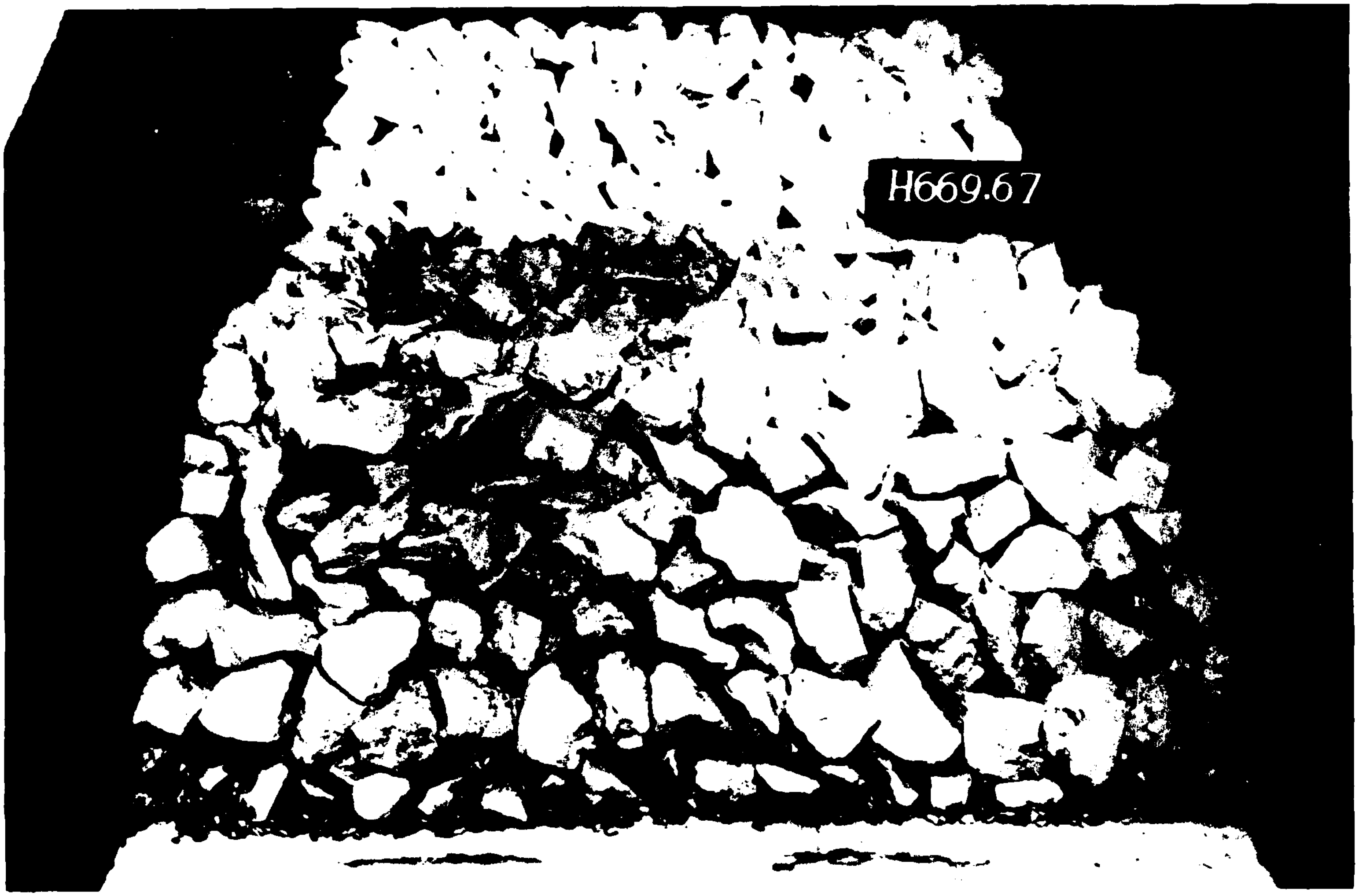


Photo 30. Landside view of Plan 5 before testing

Photo 31. Side view of Plan 5 after testing

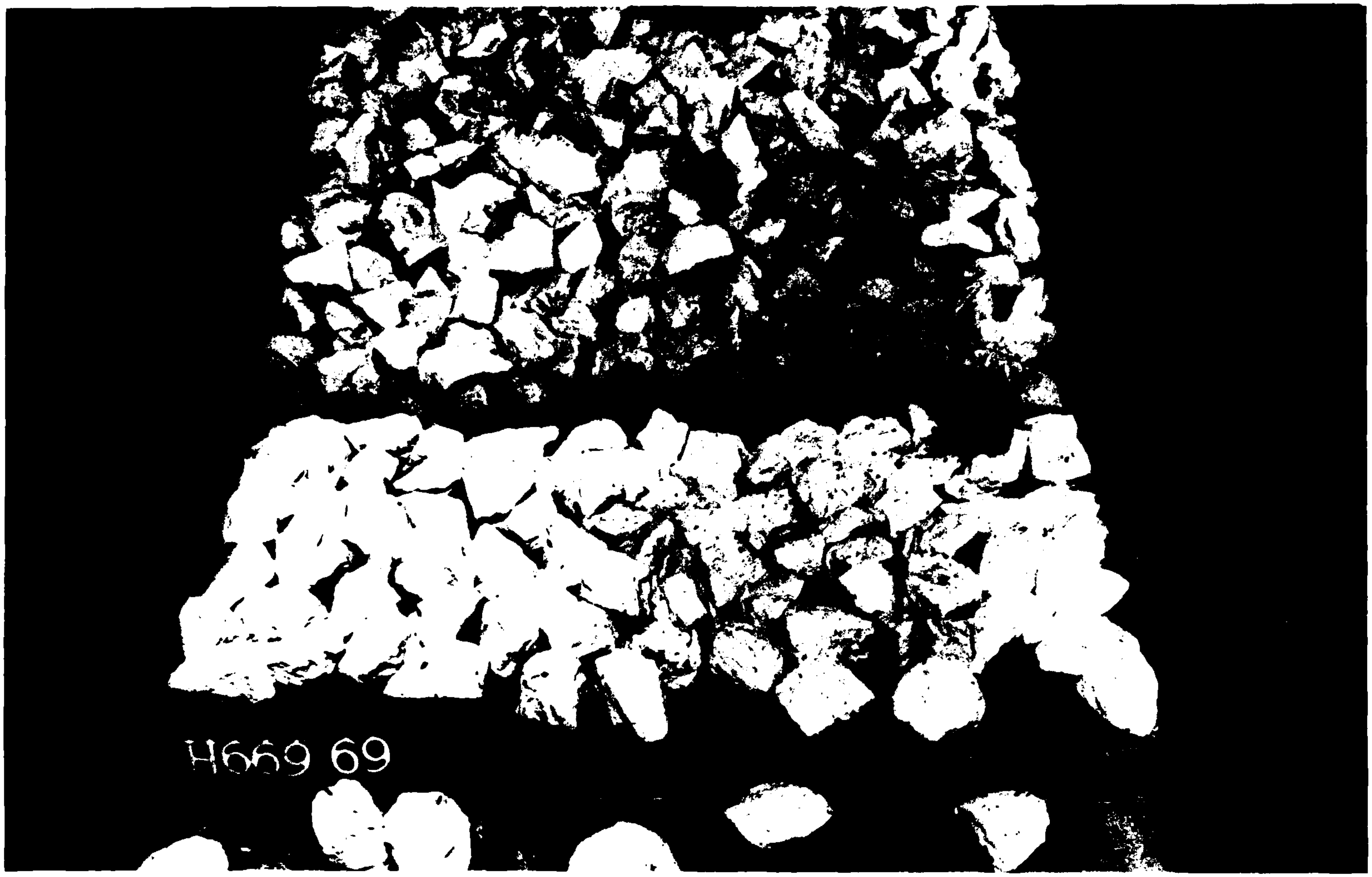


Photo 32. Sea-side view of Plan 5 after testing

Photo 33. Landside view of Plan 5 after testing

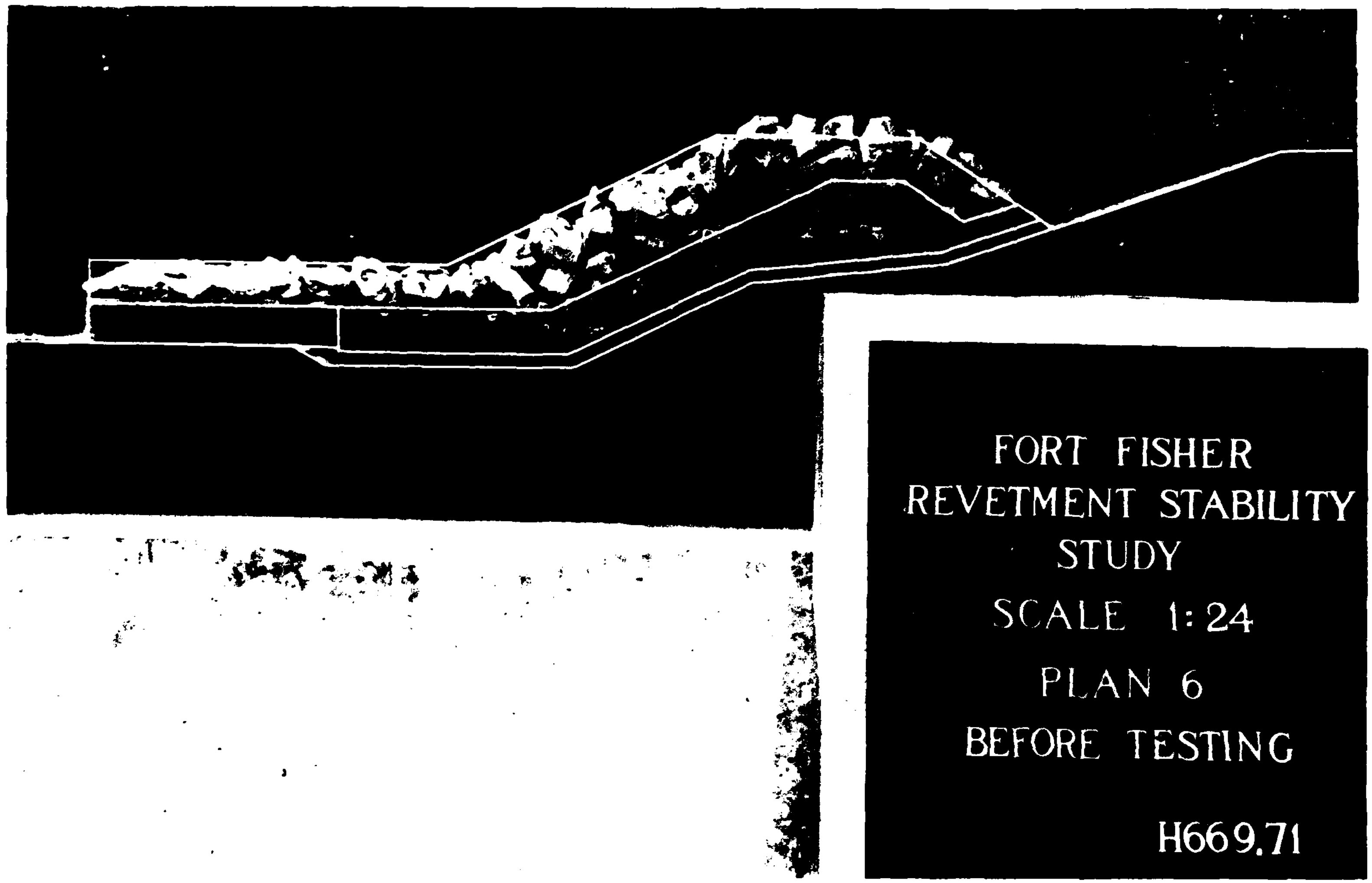


Photo 34. Side view of Plan 6 before testing

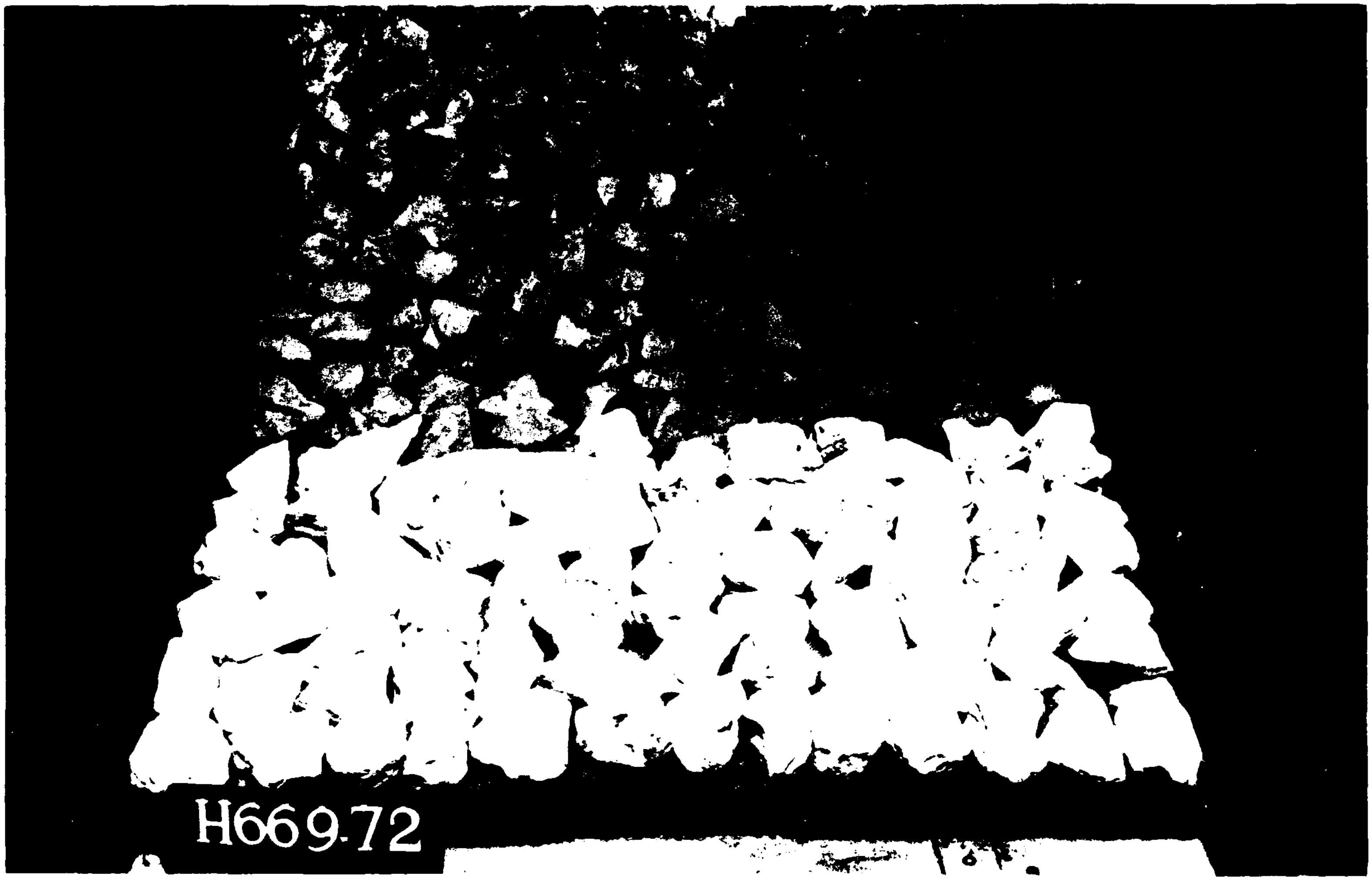


Photo 35. Sea-side view of Plan 6 before testing

Photo 36. Landside view of Plan 6 before testing

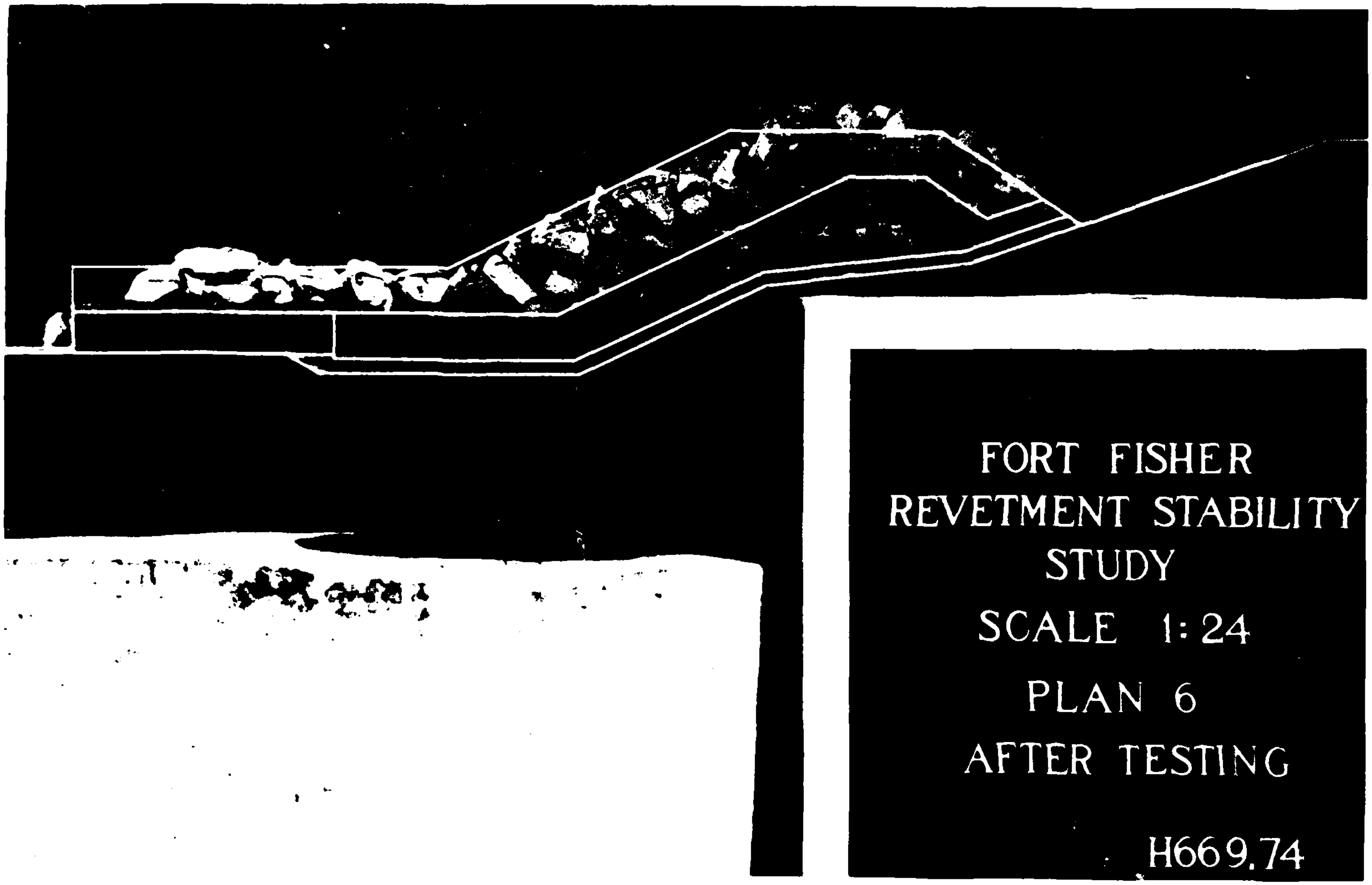


Photo 37. Side view of Plan 6 after testing

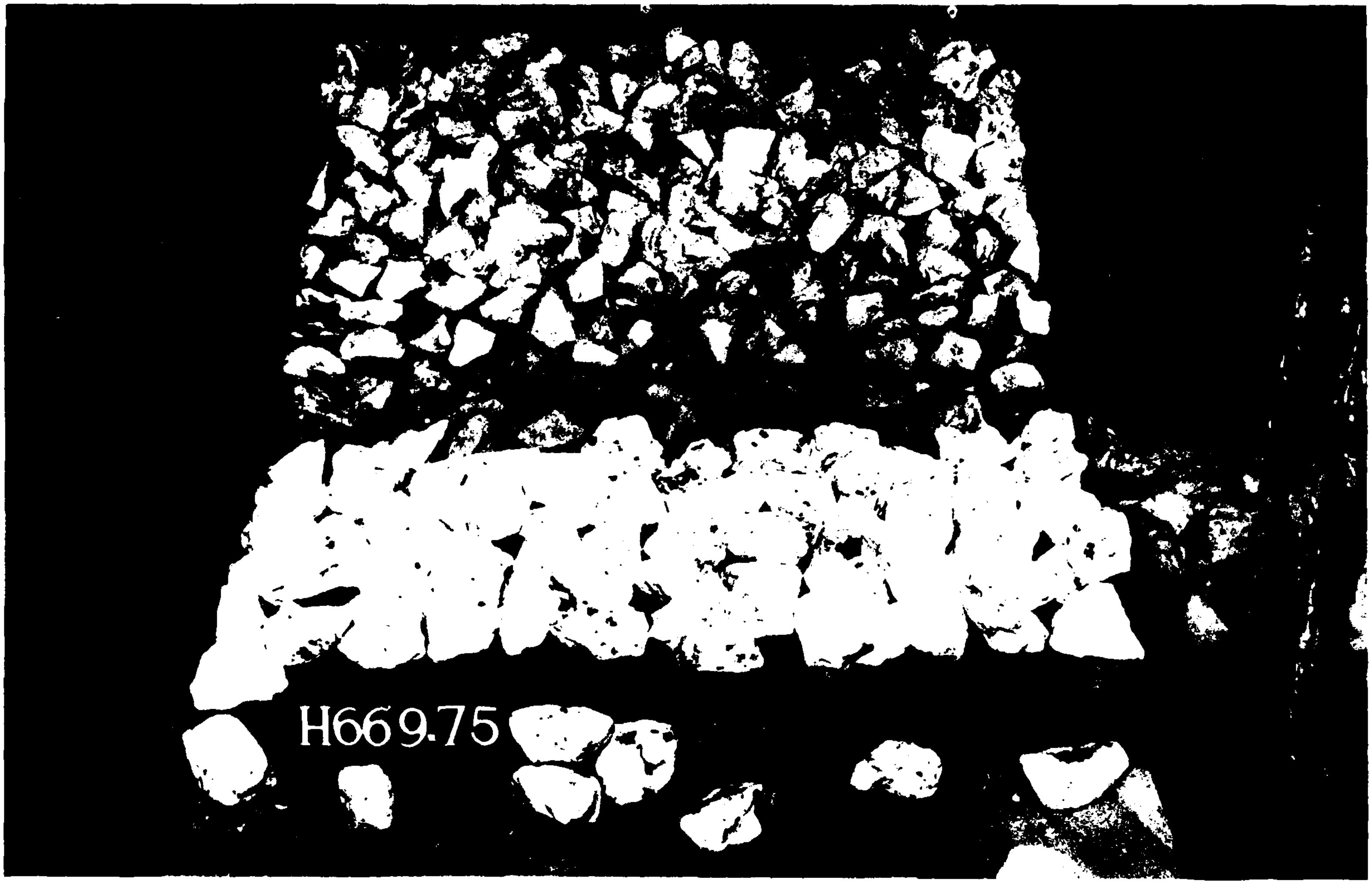


Photo 38. Sea-side view of Plan 6 after testing

Photo 39. Landside view of Plan 6 after testing

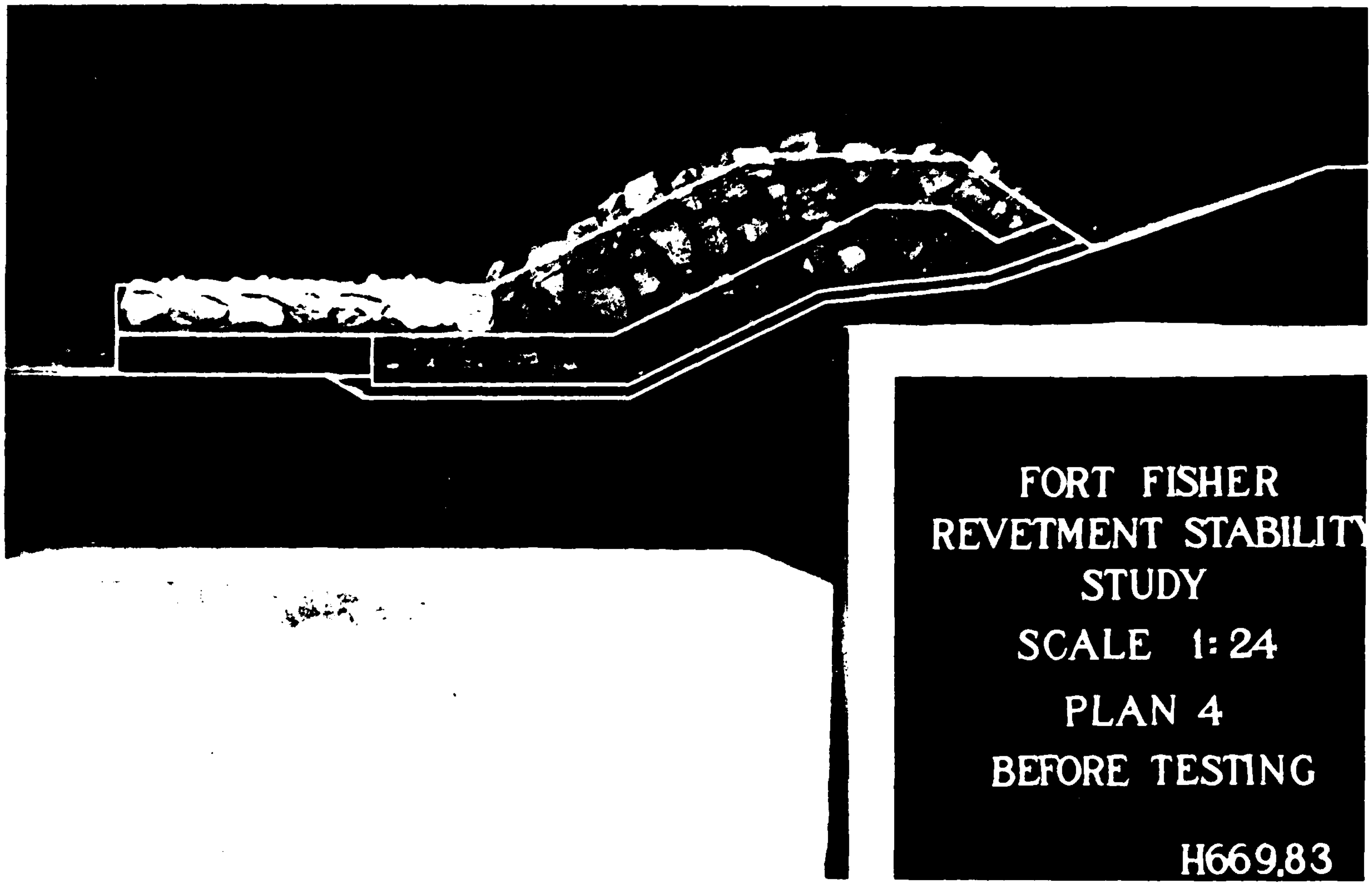


Photo 40. Side view of Plan 4 before testing, high-water-level test

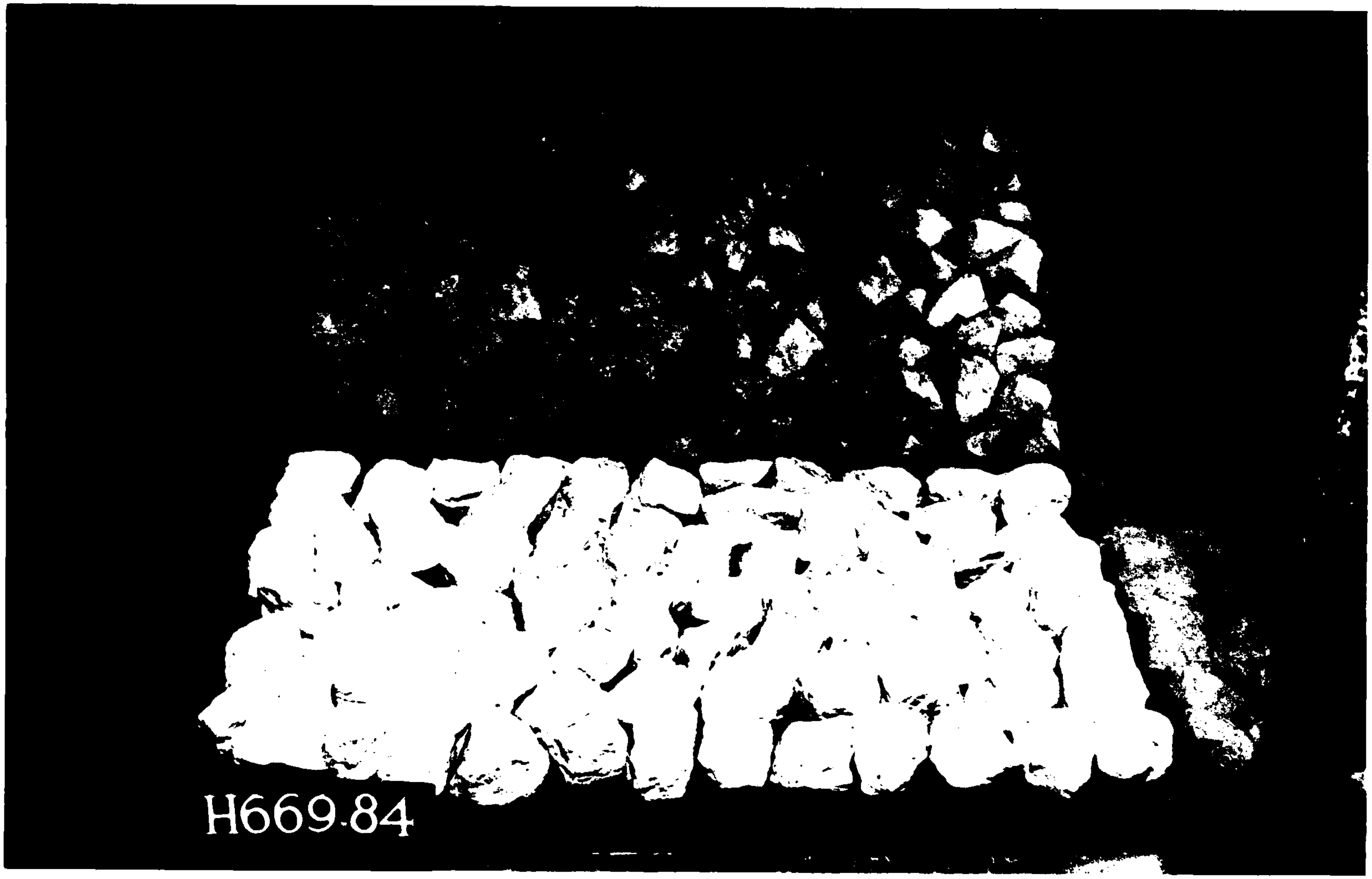


Photo 41. Sea-side view of Plan 4 before testing, high-water-level test

Photo 42. Landside view of Plan 4 before testing, high-water-level test

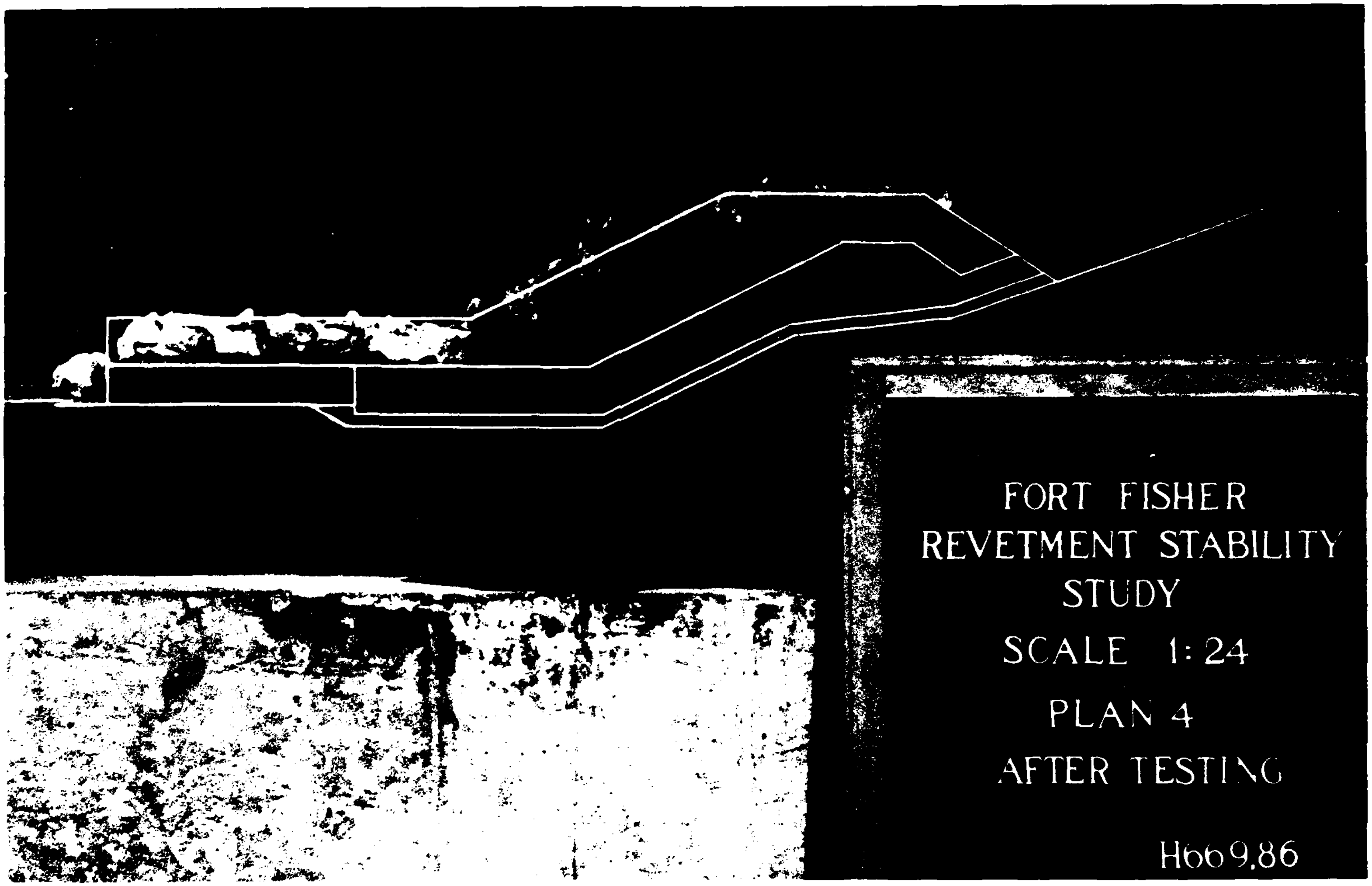


Photo 43. Side view of Plan 4 after testing Hydrographs A and C, high-water-level test

Photo 44. Sea-side view of Plan 4 after testing Hydrographs A and C, high-water-level test

Photo 45. Landside view of Plan 4 after testing Hydrographs A and C, high-water-level test

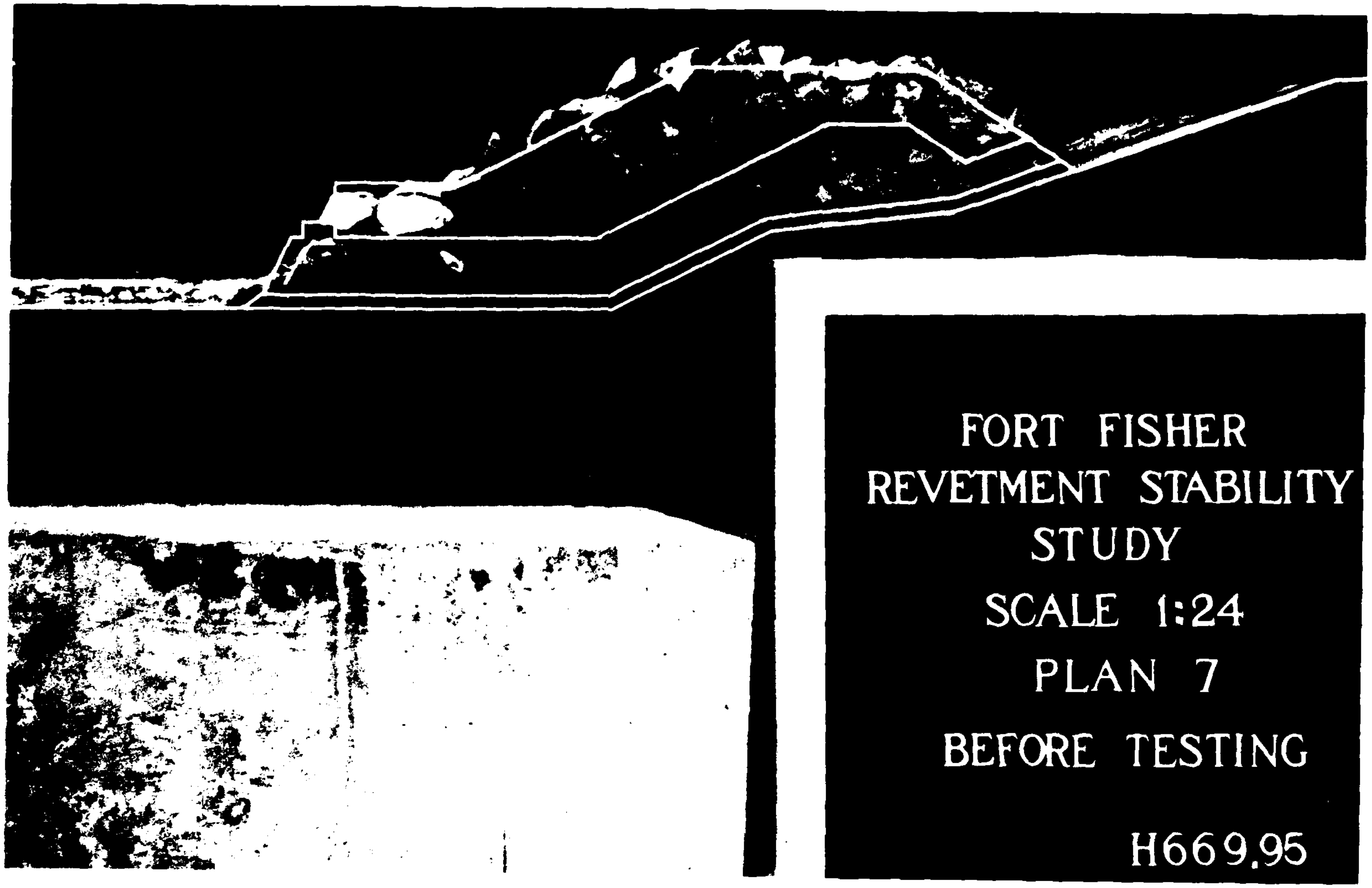


Photo 46. Side view of Plan 7 before testing

Photo 47. Sea-side view of Plan 7 before testing

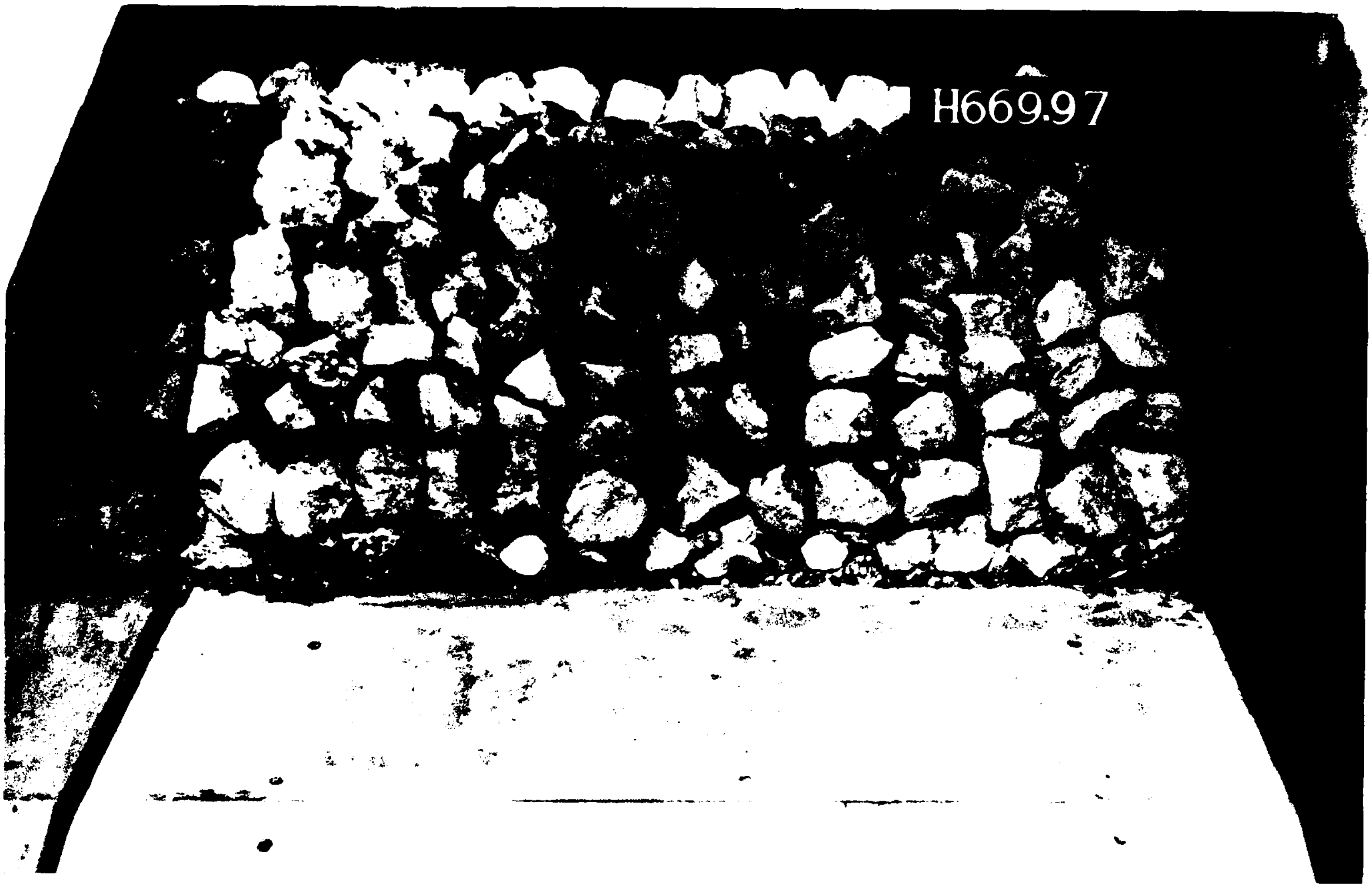


Photo 48. Landside view of Plan 7 before testing

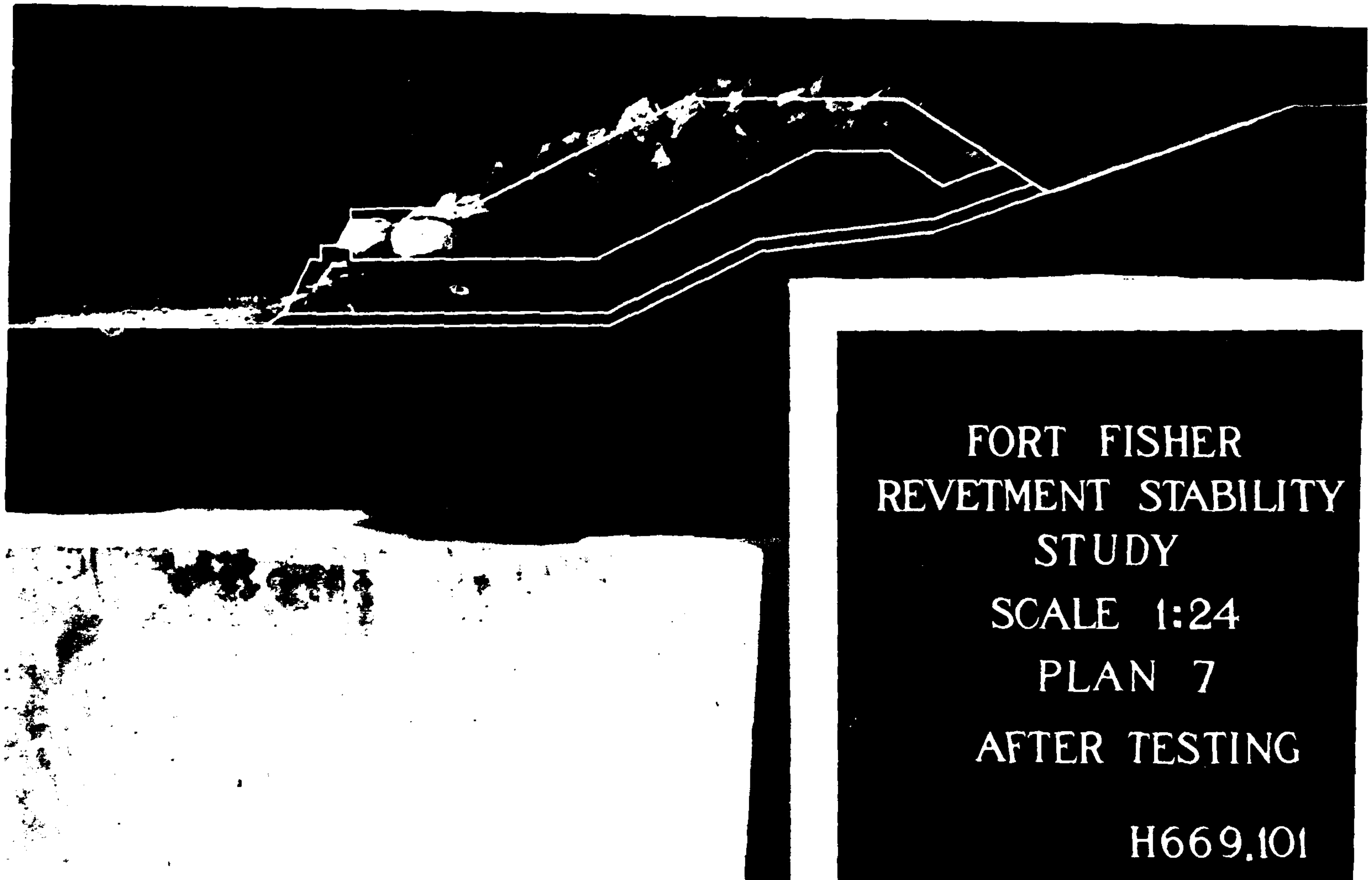


Photo 49. Side view of Plan 7 after testing Hydrograph A

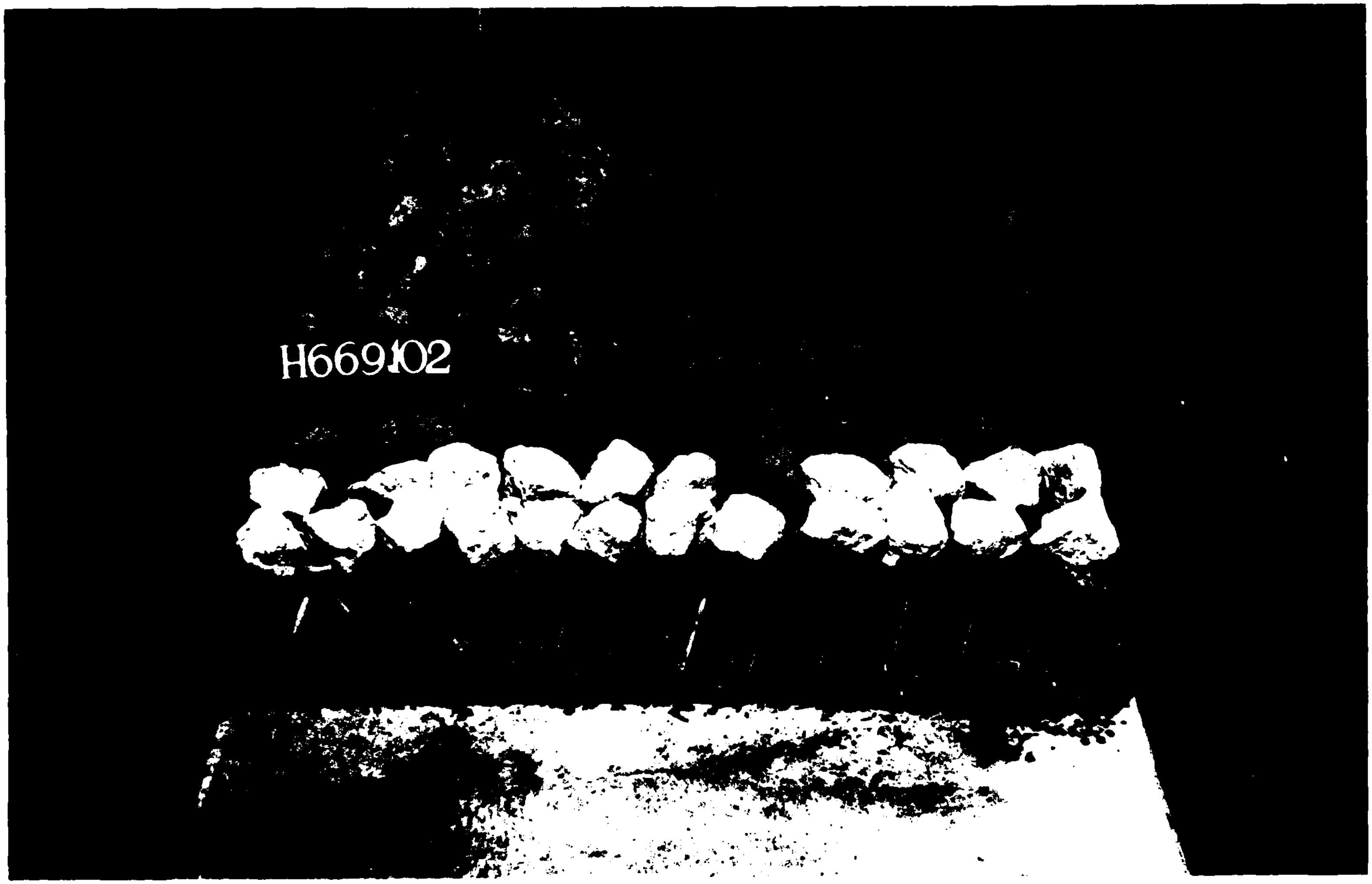


Photo 50. Sea-side view of Plan 7 after testing Hydrograph A

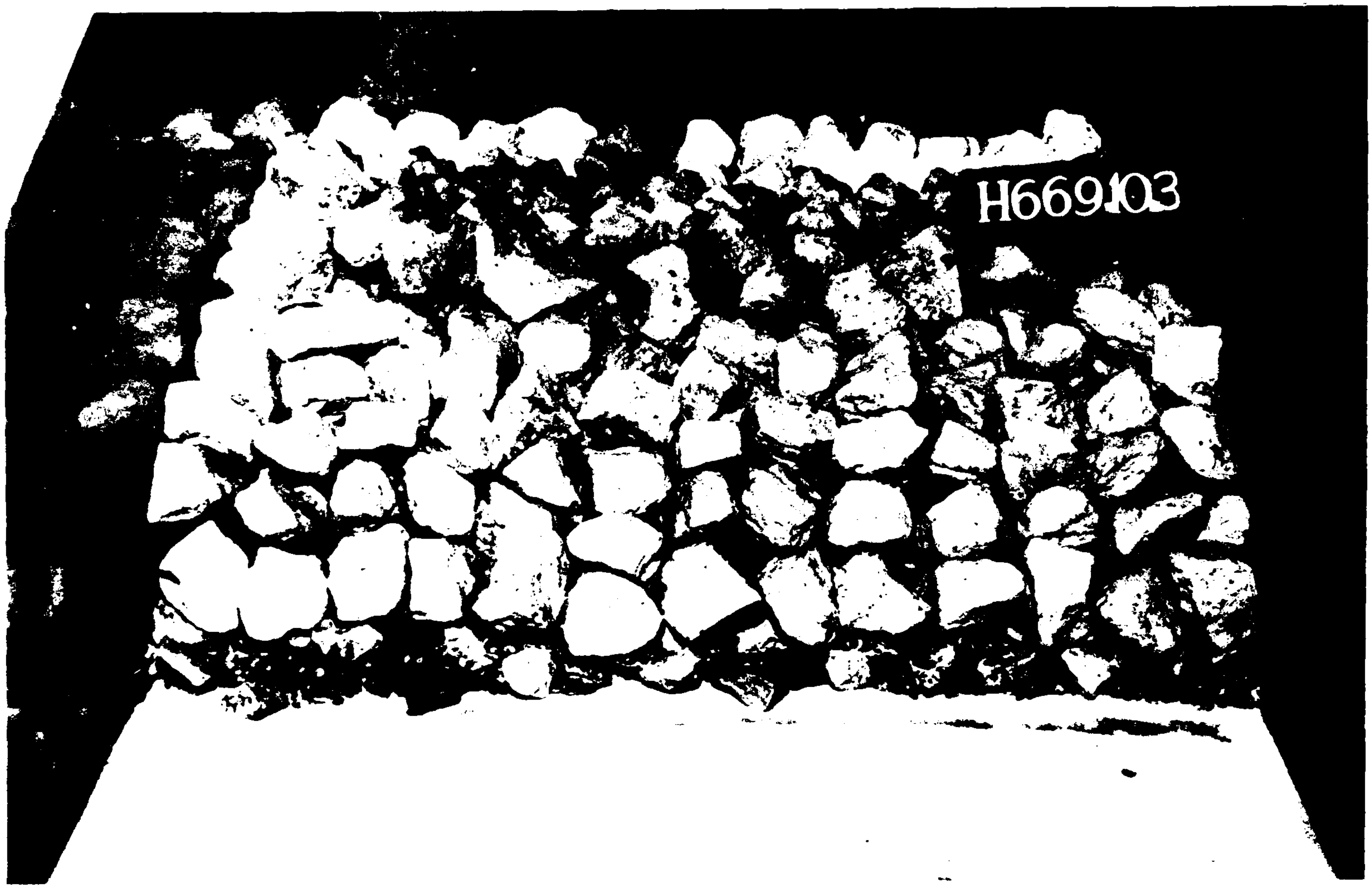


Photo 51. Landside view of Plan 7 after testing Hydrograph A

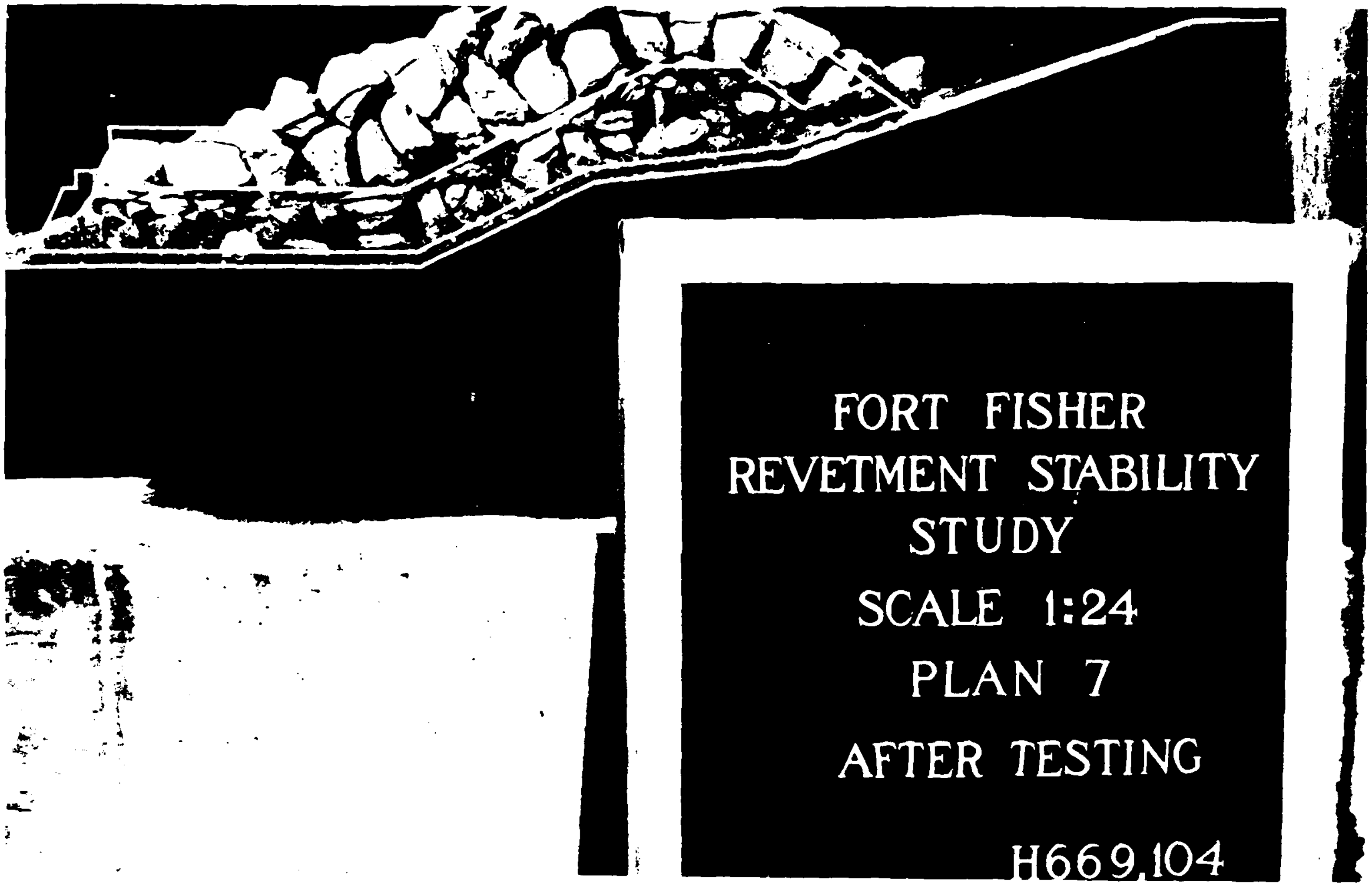
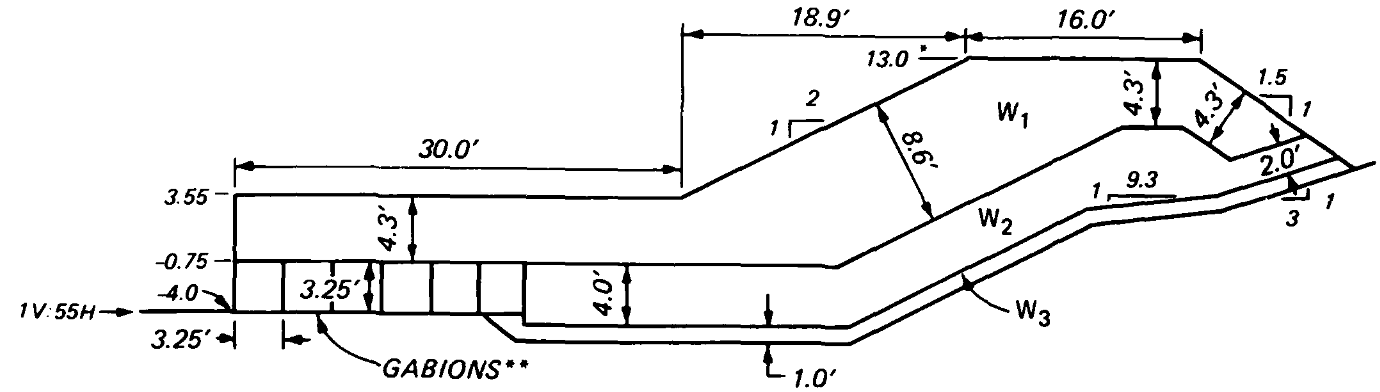


Photo 52. Side view of Plan 7 after testing Hydrographs A and C, high-water-level test

Photo 53. Sea-side view of Plan 7 after testing Hydrographs A and C, high-water-level test

Photo 54. Landside view of Plan 7 after testing Hydrographs A and C, high-water-level test

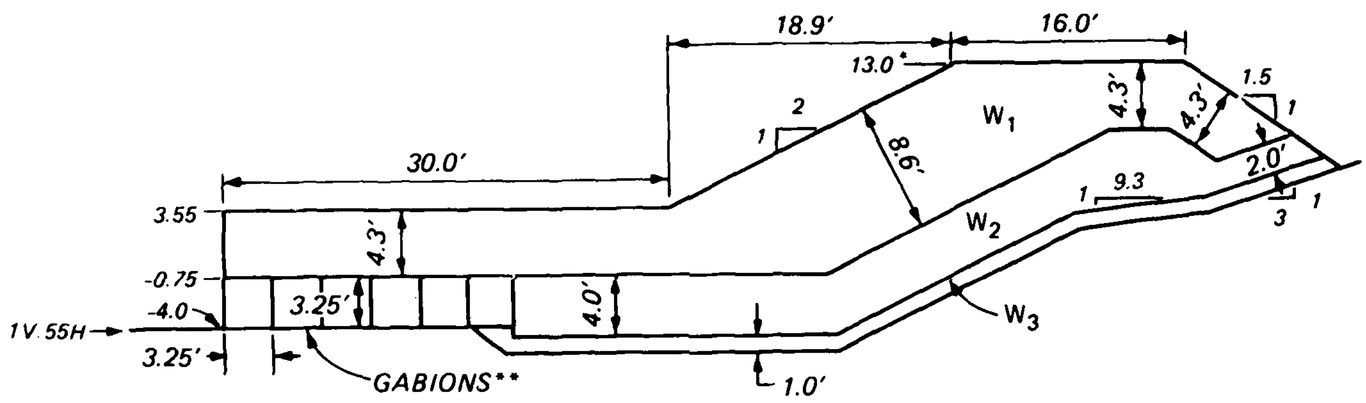


MATERIAL CHARACTERISTICS

MODEL	PROTOTYPE
† W_1 = 0.550-LB ROCK @ 165 PCF	W_1 = 8,600-LB ROCK @ 165 PCF
W_2 = 0.064-LB ROCK @ 165 PCF	W_2 = 675-LB ROCK @ 125 PCF
W_3 = 0.0002-LB ROCK @ 165 PCF	W_3 = 2-LB ROCK @ 125 PCF

* ELEVATIONS ARE IN FEET REFERRED TO NGVD
** GABIONS (3.25' x 3.25' x 9.67') FILLED WITH 1' DIAMETER MARINE LIMESTONE
† SPECIAL-PLACED CONSTRUCTION

PLAN 1

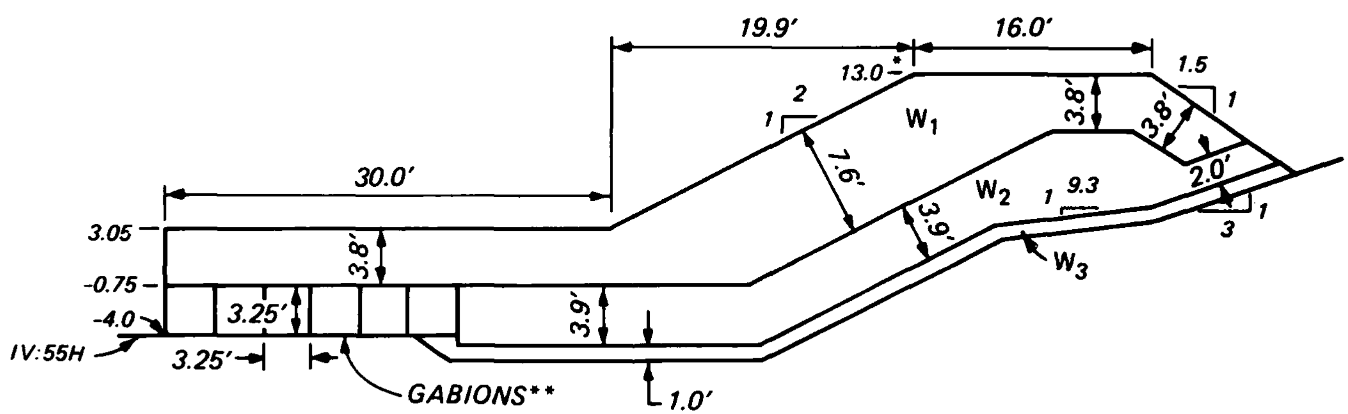


MATERIAL CHARACTERISTICS

MODEL	PROTOTYPE
† W_1 = 0.550-LB ROCK @ 165 PCF	W_1 = 8,600-LB ROCK @ 165 PCF
W_2 = 0.064-LB ROCK @ 165 PCF	W_2 = 675-LB ROCK @ 125 PCF
W_3 = 0.0002-LB ROCK @ 165 PCF	W_3 = 2-LB ROCK @ 125 PCF

* ELEVATIONS ARE IN FEET REFERRED TO NGVD
** GABIONS (3.25' x 3.25' x 9.67') FILLED WITH 1' DIAMETER MARINE LIMESTONE
† RANDOM-PLACED CONSTRUCTION

PLAN 2

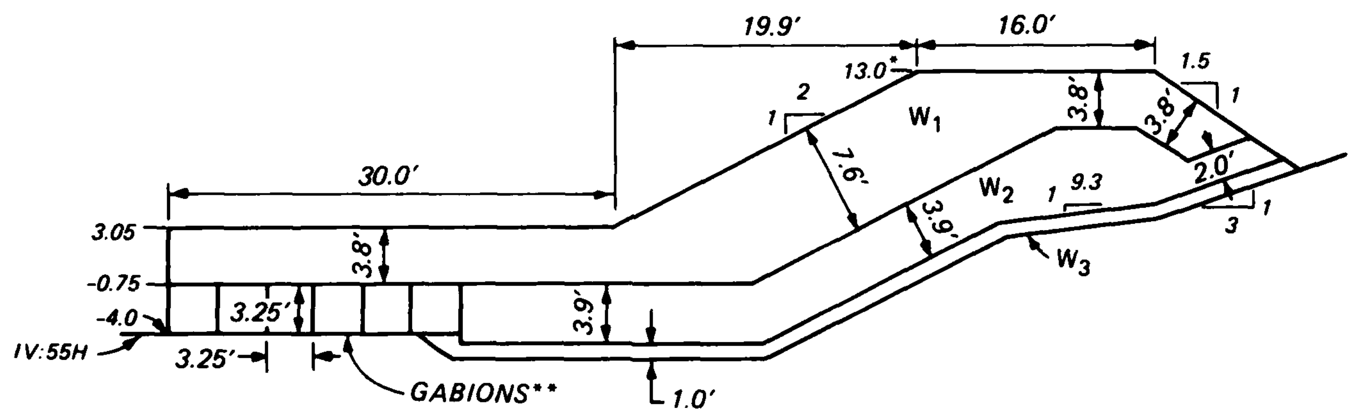


MATERIAL CHARACTERISTICS

MODEL	PROTOTYPE
† W_1 = 0.380-LB ROCK @ 165 PCF	W_1 = 5,900-LB ROCK @ 165 PCF
W_2 = 0.056-LB ROCK @ 165 PCF	W_2 = 590-LB ROCK @ 125 PCF
W_3 = 0.0002-LB ROCK @ 165 PCF	W_3 = 2-LB ROCK @ 125 PCF

* ELEVATIONS ARE IN FEET REFERRED TO NGVD
** GABIONS (3.25' x 3.25' x 9.67') FILLED WITH 1' DIAMETER MARINE LIMESTONE
† RANDOM-PLACED CONSTRUCTION

PLAN 3

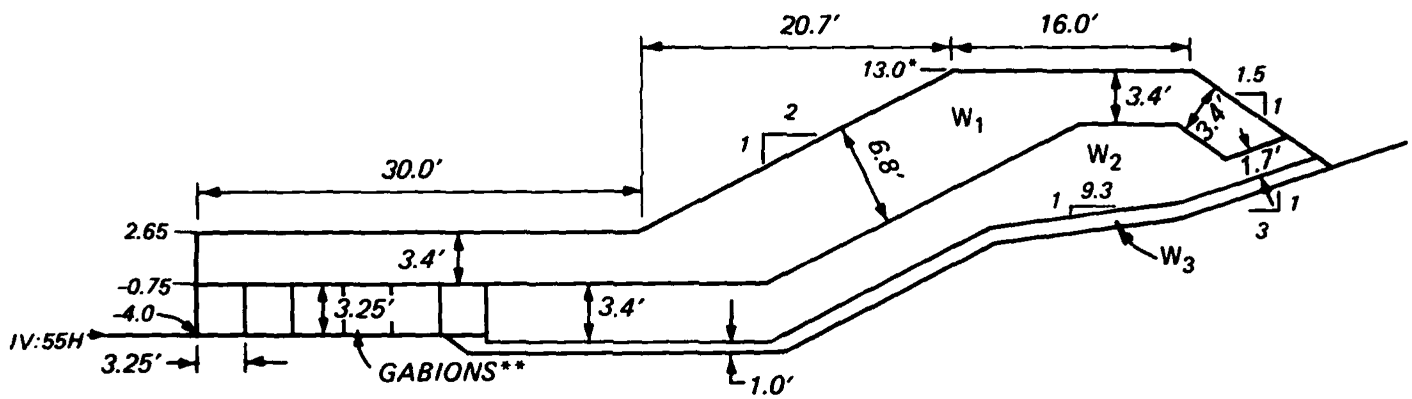


MATERIAL CHARACTERISTICS

MODEL	PROTOTYPE
† W_1 = 0.380-LB ROCK @ 165 PCF	W_1 = 5,900-LB ROCK @ 165 PCF
W_2 = 0.056-LB ROCK @ 165 PCF	W_2 = 590-LB ROCK @ 125 PCF
W_3 = 0.0002-LB ROCK @ 165 PCF	W_3 = 2-LB ROCK @ 125 PCF

* ELEVATIONS ARE IN FEET REFERRED TO NGVD
** GABIONS (3.25' x 3.25' x 9.67') FILLED WITH 1' DIAMETER MARINE LIMESTONE
† SPECIAL-PLACED CONSTRUCTION

PLAN 4

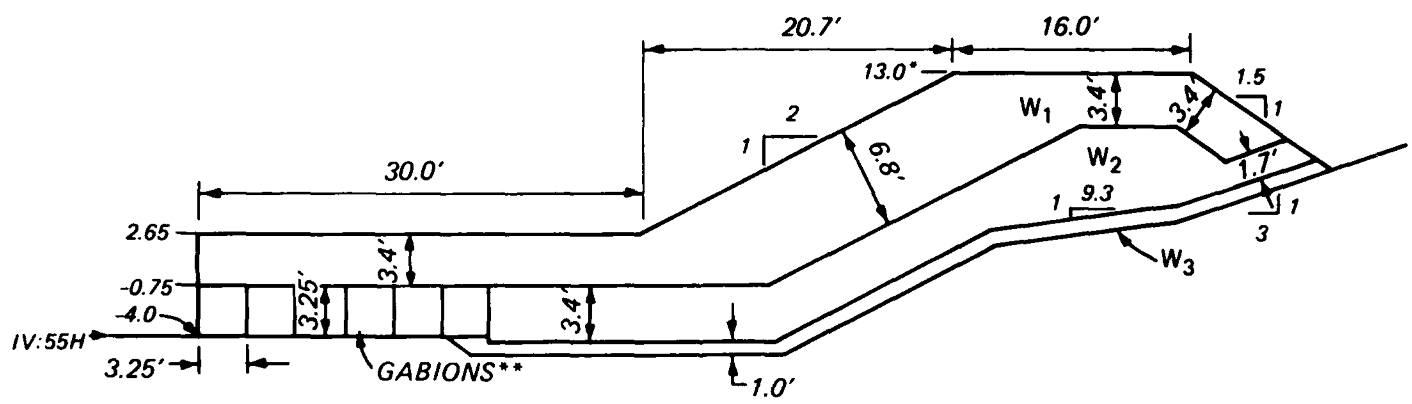


MATERIAL CHARACTERISTICS

MODEL	PROTOTYPE
† W_1 = 0.269-LB ROCK @ 165 PCF	W_1 = 4,200-LB ROCK @ 165 PCF
W_2 = 0.040-LB ROCK @ 165 PCF	W_2 = 420-LB ROCK @ 125 PCF
W_3 = 0.0002-LB ROCK @ 165 PCF	W_3 = 2-LB ROCK @ 125 PCF

* ELEVATIONS ARE IN FEET REFERRED TO NGVD
** GABIONS (3.25' x 3.25' x 9.67') FILLED WITH 1' DIAMETER MARINE LIMESTONE
† RANDOM-PLACED CONSTRUCTION

PLAN 5

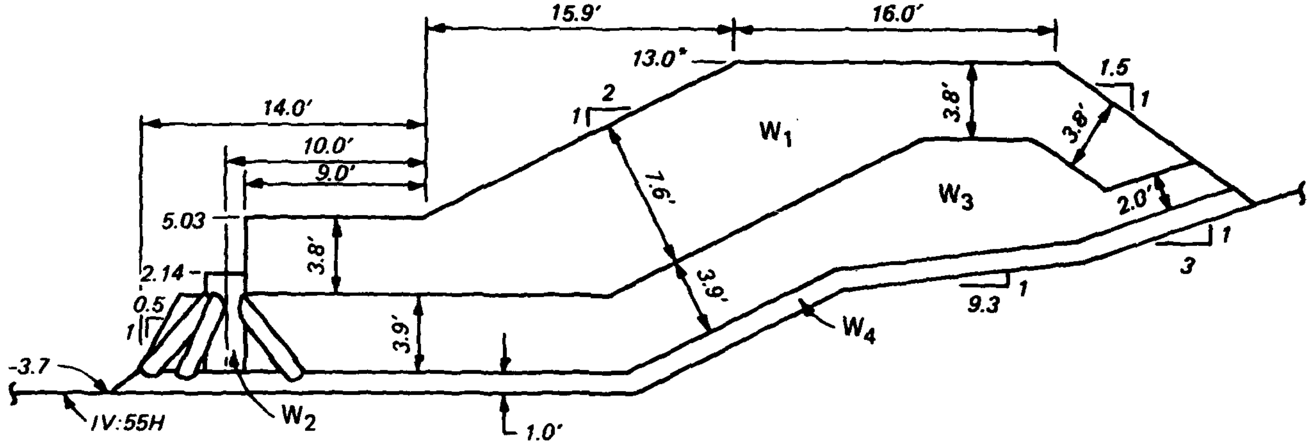


MATERIAL CHARACTERISTICS

MODEL	PROTOTYPE
† W_1 = 0.269-LB ROCK @ 165 PCF	W_1 = 4,200-LB ROCK @ 165 PCF
W_2 = 0.040-LB ROCK @ 165 PCF	W_2 = 420-LB ROCK @ 125 PCF
W_3 = 0.0002-LB ROCK @ 165 PCF	W_3 = 2-LB ROCK @ 125 PCF

* ELEVATIONS ARE IN FEET REFERRED TO NGVD
** GABIONS (3.25' x 3.25' x 9.67') FILLED WITH 1' DIAMETER MARINE LIMESTONE
† SPECIAL-PLACED CONSTRUCTION

PLAN 6



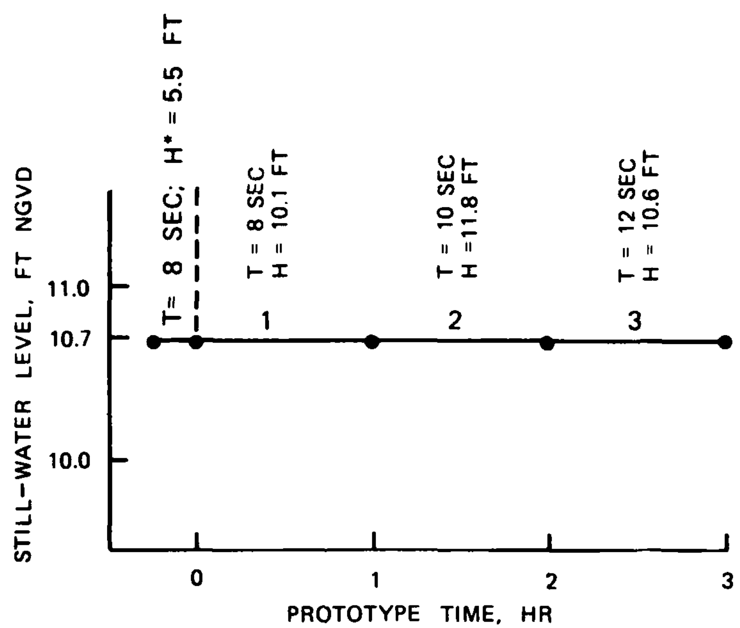
MATERIAL CHARACTERISTICS

MODEL	PROTOTYPE
**W_1 = 0.380-LB ROCK @ 165 PCF	W_1 = 5,900-LB ROCK @ 165 PCF
W_2 = 0.370-LB STA-PODS @ 167 PCF	W_2 = 8,919-LB STA-PODS @ 150 PCF
W_3 = 0.056-LB ROCK @ 165 PCF	W_3 = 590-LB ROCK @ 125 PCF
W_4 = 0.0002-LB ROCK @ 165 PCF	W_4 = 2-LB ROCK @ 125 PCF

* ELEVATIONS ARE IN FEET REFERRED TO NGVD

** SPECIAL-PLACED CONSTRUCTION

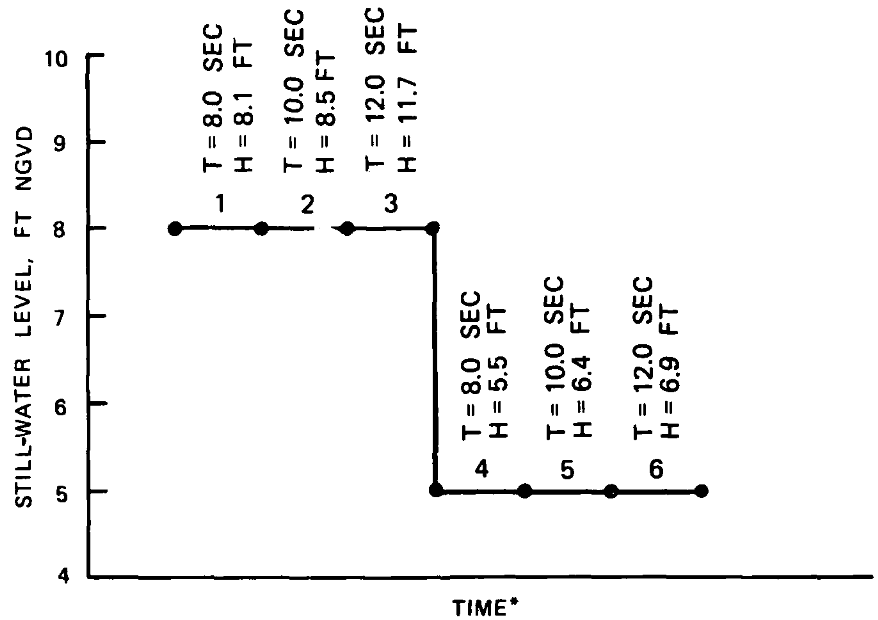
PLAN 7



LEGEND

T WAVE PERIOD
H* SHAKEDOWN WAVE HEIGHT
H WORST BREAKING WAVE HEIGHT
1-3 HYDROGRAPH STEPS

HYDROGRAPH A

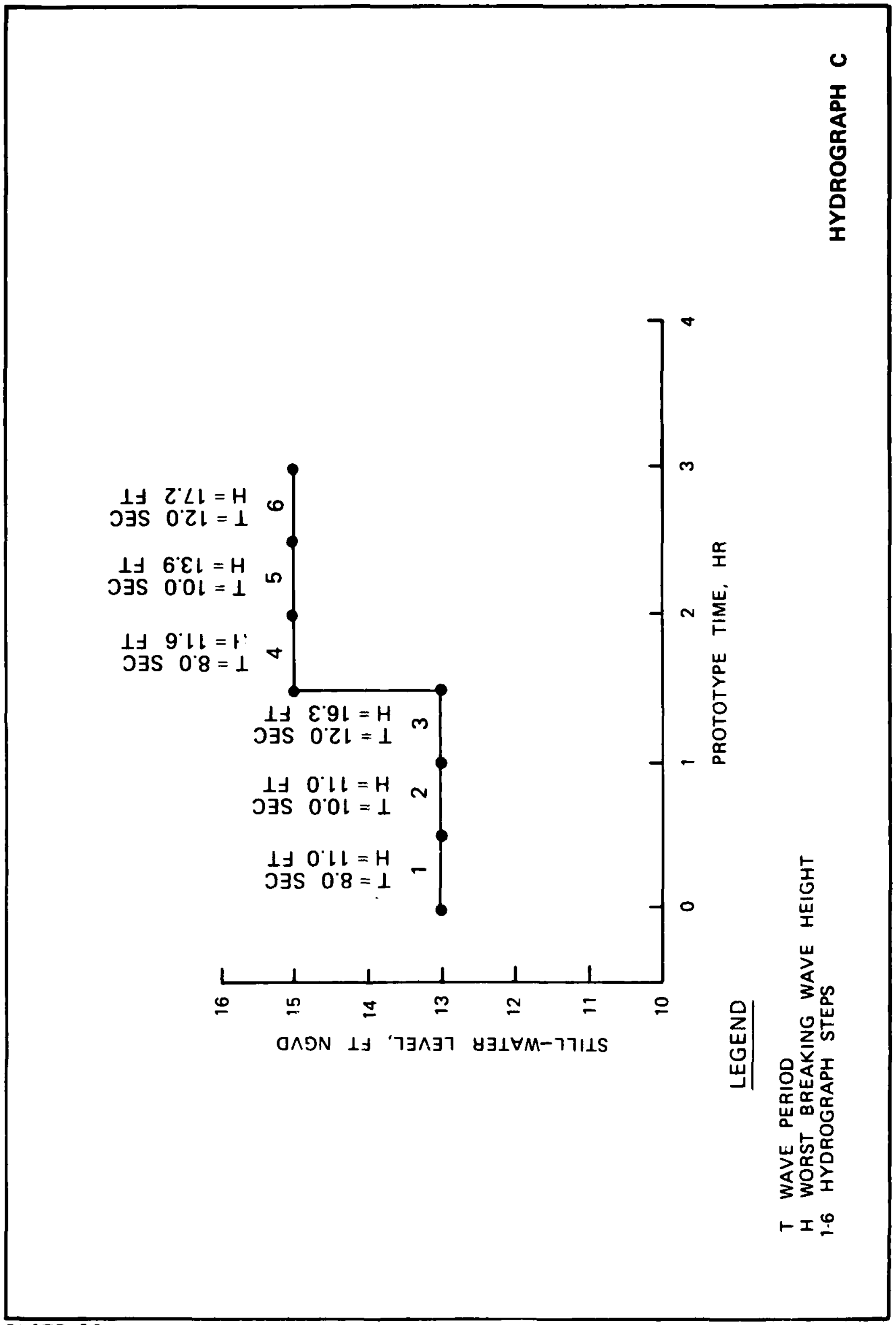


LEGEND

T, WAVE PERIOD
H, WORST BREAKING WAVE HEIGHT
1-6, HYDROGRAPH STEPS

THERE WAS NO FIXED TEST DURATION . EACH STEP WAS RUN FOR A SUFFICIENT DURATION OF TIME TO BE SURE THAT EITHER NO DAMAGE WAS GOING TO OCCUR, OR THAT NO ADDITIONAL DAMAGE WOULD OCCUR.

HYDROGRAPH B



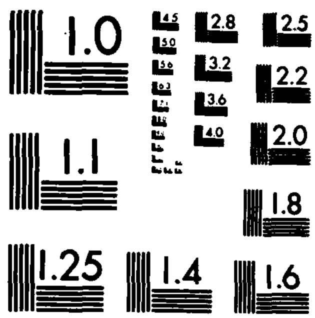
HYDROGRAPH C
STILL-WATER LEVEL, FT NGVD
16
15
14
13
12
11
10
PROTOTYPE TIME, HR
0
1
2
3
4
1
T = 8.0 SEC
H = 11.0 FT
2
T = 10.0 SEC
H = 11.0 FT
3
T = 12.0 SEC
H = 16.3 FT
4
T = 8.0 SEC
H = 11.6 FT
5
T = 10.0 SEC
H = 13.9 FT
6
T = 12.0 SEC
H = 17.2 FT
LEGEND
T WAVE PERIOD
H WORST BREAKING WAVE HEIGHT
1-6 HYDROGRAPH STEPS

AD-A123 754 REVETMENT STABILITY STUDY FORT FISHER STATE HISTORIC SITE NORTH CAROLINA..(U) ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MS HYDRA.. D G MARKLE

2/2

UNCLASSIFIED NOV 82 WES/TR/HL-82-26 F/G 13/2 NL

END
DATE
FILMED
83
DTIC



MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

APPENDIX A: NOTATION

A	Area, ft^2
H	Wave height, ft
k_Δ	Armor unit layer thickness coefficient
ℓ	Characteristic length of armor stone, ft
L	Length, linear scale, ft
NGVD	National Geodetic Vertical Datum (formerly mean sea level)
S	Specific gravity
T	Time
V	Volume, ft^3
W	Weight, lb
γ	Specific weight, pcf

Subscripts

a	Refers to armor units
m	Refers to model quantities
p	Refers to prototype quantities
r	Refers to ratio of model quantities to prototype quantities (i.e., r = m/p)
w	Refers to water
1-4	Refers to different stone or armor unit sizes

In accordance with letter from DAEN-RDC, DAEN-ASI dated 22 July 1977, Subject: Facsimile Catalog Cards for Laboratory Technical Publications, a facsimile catalog card in Library of Congress MARC format is reproduced below.

Markle, Dennis G.
Revetment stability study Fort Fisher State Historic Site, North Carolina : Hydraulic Model Investigation / by Dennis G. Markle (Hydraulics Laboratory, U.S. Army Engineer Waterways Experiment Station). -- Vicksburg, Miss. : The Station ; Springfield, Va. ; available from NTIS, 1982.
27, [2] p., 54 photos, 10 p. of plates : ill. ; 27 cm. -- (Technical report ; HL-82-26)
Cover title.
"November 1982."
Final report.
"Prepared for U.S. Army Engineer District, Wilmington."

1. Fort Fisher (N.C.) 2. Hydraulic models. 3. Hydraulic structures. 4. Shore protection. I. United States. Army. Corps of Engineers. Wilmington District.

Markle, Dennis G.
Revetment stability study Fort Fisher State : ... 1982.
(Card 2)

II. U.S. Army Engineer Waterways Experiment Station. Hydraulics Laboratory. III. Title IV. Series: Technical report (U.S. Army Engineer Waterways Experiment Station) ; HL-82-26.
TA7.W34 no.HL-82-26